ANNEX C - THE SANDWIP CROSS-DAM

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ANNEX C
THE SANDWIP CROSS-DAM

C.1 Introduction

The delta of the Ganges, Brahmaputra and Meghna rivers, forming a large part of Bangladesh, is practically unprotected and subject to large morphological changes due to considerable erosion of the outer banks of these channels and accretion at the inner banks of these channels and the lee side of islands and chars.

Some 40 years ago the Lower Meghna still bifurcated just north of Bhola island: the eastern branch discharged into the Sandwip and Hatia channels at that time, whereas the other, southern branch mainly discharged into the Shahbazpur river, with a secondary connection, north of Hatia island, discharging into Hatia channel. Since then the eastern branch of the Lower Meghna has silted up and finally lost its discharge function completely after the construction in 1957 of the Meghna cross-dam no 1. The construction of this dam accelerated the siltation process in this river branch, and a second dam (cross-dam no. 2) was built approximately 18 km further downstream in 1964. After that, huge areas have accreted south-east of the Noakhali mainland and further closures have been realized, such as the closure of the Daria Nadi by the Land Reclamation Project (LRP), which stopped erosion of old land and stimulated the accretion of a large area of new land.

After the siltation and disappearance of the eastern branch of the Lower Meghna, Sandwip channel lost its significance for the discharge of the upland river floods and now acts purely as a tidal estuary. The Shahbazpur-Hatia channel system, on the other hand, acts as a tidal river with very high monsoon discharges. These two systems are connected by cross-channels between Sandwip island and the Noakhali mainland.

The tidal waves propagating through the Sandwip and Hatia channels from the south meet each other in these cross-channels. Since the tidal wave through Sandwip channel reaches the cross-channels earlier than the one through Hatia channel, due to differences in distortion of both tidal waves, the water-levels on both sides of the cross-channels generally are not the same, resulting in tidal exchange flows through these channels. The flow through the cross-channel between Sandwip island and Char Pir Baksh is concentrated near the bank of Sandwip and causes considerable erosion of the north-west coast of the island, while accretion occurs at the Char Pir Baksh side. The west coast of Char Pir Baksh however is presently eroding.

As indicated by the pre-feasibility study undertaken in December 1983 (See LRP technical report no. 19, December 1984) the best method to arrest the erosion of the north-west coast of Sandwip island would be to close off the cross-channels between Sandwip and the Noakhali mainland by constructing a cross-dam. As part of the present feasibility study on the Sandwip cross-dam development scheme (which includes the reclamation and development of already existing and newly accreted land), a study was made of possible closure methods, resulting in a (preliminary) design for the dam at feasibility level, proposals for the execution of the works and cost estimates. A detailed description of these aspects is presented in this annex. The design of the cross-dam applies in particular to those sections which cross the western channel (between Char Lakhi and Char Pir Baksh) and the eastern channel (between Char Pir Baksh and Sandwip). Furthermore, much attention will be given to the closure methodology and the rationale for selecting closure methods.

The actual closures concern four tidal channels which cross the alignment of the dam. (A preliminary alignment is given in Figure C.1). The channels to be closed are:

- Western channel, between Char Lakhi and Char Pir Baksh (5250 m wide)
- Eastern channel, between Char Pir Baksh and Sandwip (8000 m wide)
- Two central channels, located east of Char Pir Baksh (2000 m and 1000 m wide)

It should be realised that in making preliminary designs certain assumptions had to be made, which will have to be verified in the final design stage. For this verification additional data collection and laboratory investigations are necessary, as detailed in Chapter C.16. The designs presented here are based on the functional requirements listed in the following subsection.

C.1.1 Functional requirements

The principal hydraulic functions of the Sandwip cross-dam are, firstly, to halt the erosion of the north-western coast of Sandwip island and, secondly, to create favourable conditions for sedimentation and land accretion. A cross-dam between the Noakhali mainland and Sandwip, as envisaged in this study, will also stop the erosion of the western coast of Char Pir Baksh.

The structural requirements following from these functions (see Section C.1.2) and the further elaboration of the design result in a dam with a crest width of 3 m and a crest level of the dam in the closure sections which has been chosen in such a way that the dam may be overtopped with a frequency of occurrence of about fiteen times per year (see Sections C.2.4. and C.11.1). The dam section on Char Dir Baksh will become part of the future polder dike system and the crest level of this section has been fixed in accordance with the design criteria used for these dikes (see Annex F, Section F.4.3).

It has been suggested that the cross-dam should also provide a road connection between Sandwip island and the Noakhali mainland. To fulfill this function, the dimensions of the cross-dam would have to be adapted.

Although no firm design criteria have been mentioned, it seems reasonable to adopt a road width of 4.5 m and a crest width of 6 m as minimum measures for the dam on which a road has to be built.

The question arises whether or not the crest height of the cross-dam also would have to be modified, particularly in the closure sections of the western and eastern channels.

It could be argued that the road should 'always' be trafficable and 'never' be overtopped. In Section C.1.2 below it is mentioned that the crest height of the central section of the cross-dam, fixed at PD + 8.5 m, is based on a 1:20 years flood level plus a freeboard of 1.5 m for exposed areas. The crest heights of the western and eastern sections have been designed in such a way that the frequency of overtopping, taking wind set-up into account, is expected to be in the order of 15 times per year. It is also mentioned in Section C.1.2 that the additional costs for a dam with the proposed crest width of 3 m -which is not sufficient for a road- but with a crest height of PD + 8.5 m over the entire length, already amount to about Tk 1400 million.

Bearing in mind that the period during which the cross-dam is fully exposed to sea conditions is relatively short, as within a few years new land will accrete on either side, it seems justified to accept that during this period the dam will have to be closed to traffic, say, about 15 times per year for periods varying from one to several hours, or longer if repairs are necessary.

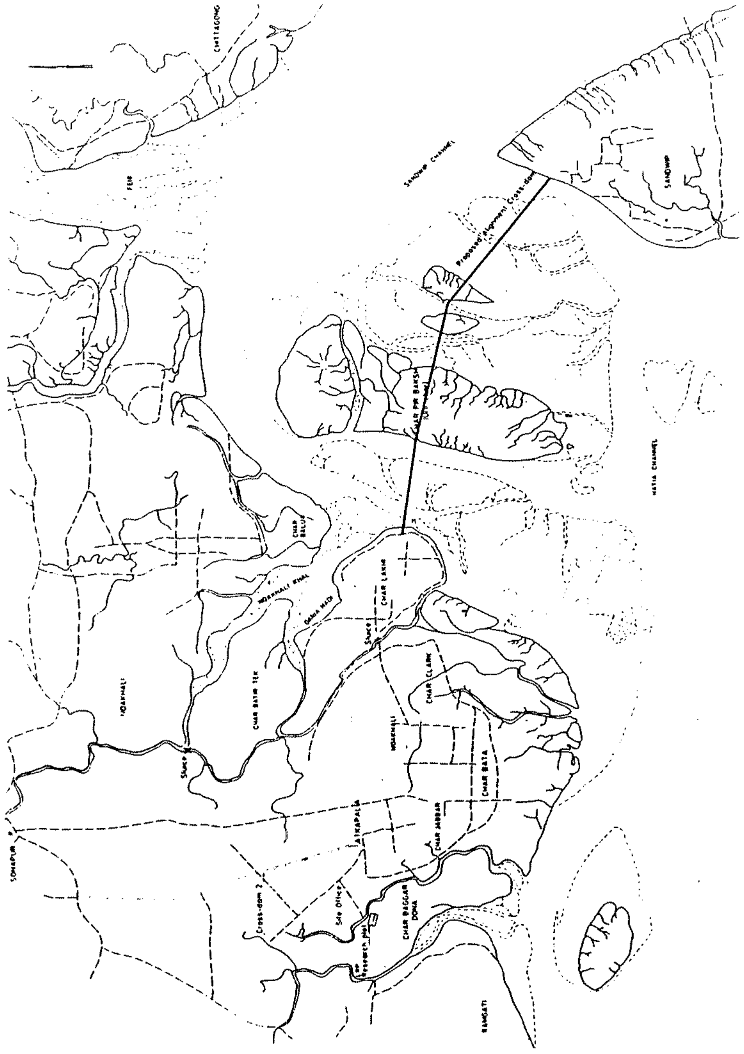
Consequently, an estimate has been made of the additional cost for a dam with the same crest heights as proposed in this report, but with a crest width of 6 m instead of 3 m. These additional costs are mainly caused by the larger volumes needed for a dam with a broader crest, and amount to about Tk 375 000 000.

Observations about the economics of such an additional investment for the benefit of a road connection between Sandwip island and the Noakhali mainland are presented in Annex G, Section G.4. There it is concluded that this additional investment cannot be justified economically, on the one hand in view of the traffic volume, and on the other hand because a road connection can be built at considerably less cost on the new land accreted on either side of the cross dam within a few years after its completion. Consequently, the design of the cross-dam in this study is based on the principal hydraulic functions mentioned above.

C.1.2 Structural requirements

The hydraulic functional requirements mentioned in Subsection C.1.1 lead to the following structural requirements:

- No currents over the dam should occur below a level coinciding with the highest astronomical high water-level with an exceedance frequency of, say, 1 %; occasional higher water-levels (caused by wind setup) which would overtop the dam with a frequency upto around fifteen times per year would not hamper the accretion process and could therefore be accepted in principle.
- Overtopping of the dam by waves can be accepted in principle, provided no (or only minor) damage would occur to the dam.



PRELIMINARY ALIGNMENT OF CROSS-DAM

scale 1:300,000

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.1

- Permeability of the dam should be small enough to restrict the exit velocity of any seepage flow.
- The structure should be stable both from the hydraulics and from the soil mechanics viewpoint.
- The lifetime of the dam should be at least 10 years (after which period its primary functions will cease to exist).
- Risk levels during and after construction should be acceptable, bearing in mind the functional requirements of the cross-dam mentioned in Subsection C.1.1.

C.1.3 Loads

The following loads should be taken into consideration in the (selection of) designs for the cross-dam:

- water-level differences at various levels
- wave loads at various water-levels
- currents over the cross-dam
- longitudinal currents
- mechanical loadings
- seismic loadings.

These loads are to be considered both during and after construction of the cross-dam.

It has been argued that the dam should be 'non-overtoppable', to eliminate the risk of damage to the newly accreted, not yet empoldered land. This condition cannot be fulfilled, but it can be said that a dam with a crest level at PD + 10 to 11 m would closely approach this condition, as the frequency of overtopping (under severe cyclone conditions) is estimated to be in the order of 1:100 to 1:500 years.

Design heights for polder embankments in the project area are based on 1:20 years flood levels, which have been determined with data series of 20 years annual maximum flood levels at Sandwip and correlations with on-site tide gauging. The effects of the cross-dam and subsequent accretion on the water-levels have been calculated with the NETFLOW and WAQUA models. Based on this approach tentatively an average 1:20 years flood level of PD + 7 m has been arrived at for the project area. A freeboard of 1.5 m for exposed areas has been added to this (standard for the BWDB Coastal Embankment Project), resulting in a crest level of PD + 8.5 m, which has also been adopted for the central section of the cross-dam, on Char Pir Baksh and across the central channels, as this section will become part of the future polder-dike system. The crest levels of the western and eastern sections of the cross-dam have been fixed, in accordance with the above-mentioned structural requirements, at PD + 6 m and PD + 5 m respectively (Figure C.5). An estimate has been made of the additional cost for a dam with the proposed crest width of 3 m, but with a crest level of PD + 8.5 m over the entire length of the cross-dam. These additional costs amount to about Tk 1 410 000 000, or about 40 % of the estimated construction cost of the proposed dam.

As mentioned above, the proposed dam may be overtopped with a frequency of occurrence of about 15 times per year. Generally, damage to the newly accreted land will be small and occur only in the immediate vicinity of the dam. It is to be expected that such damage quickly will be undone by natural sedimentation in the period following the overtopping, and, therefore, can be accepted. Heavier damage may occur during severe cyclone condtions, buth the expected frequency of occurrence of this condition is low. Moreover, soon after the completion of the cross-dam the new land will be protected by polder embankments with a crest level of PD + 8.5 m.

For the eastern closing dam, during severe cyclonic conditions a scour depth of 1.5 m at the most unfavourable location is expected, which is only over a short length along the alignment. This rather low scouring rate forms no danger for the stability of the overtoppable dam. For the western gap, scour depths of 2.5-5 m are predicted under extremely severe conditions: for this section further information has to be gathered during the final design stage on the slope angle of the scour hole based on model tests (see also Section C.8).

C.2 Basic data and boundary conditions

C.2.1 Tides and tidal currents

The tidal wave from the ocean (Bay of Bengal) approaches the project area from the south and continues through the Hatia and Sandwip channels. The tide is semi-diurnal with some daily inequality. There is a considerable variation between neap and spring tides. There is also a seasonal variation of the mean sea-level. The highest level is reached in summer and the lowest in winter. The variation is caused by long-term differences in atmospheric pressure, wind direction, salinity and river discharge (during the summer and winter periods).

The semi-diurnal, the fortnightly and the seasonal variations are related to astronomic factors. Consequently they are predictable. The nearest tidal station for which predictions are published in the Bangladesh Tide Tables (published by BIWTA) is at Satal Khal on the west coast of Sandwip. A diagram of all predicted high and low waters for 1985 is shown in Figure C.2. The graph clearly indicates

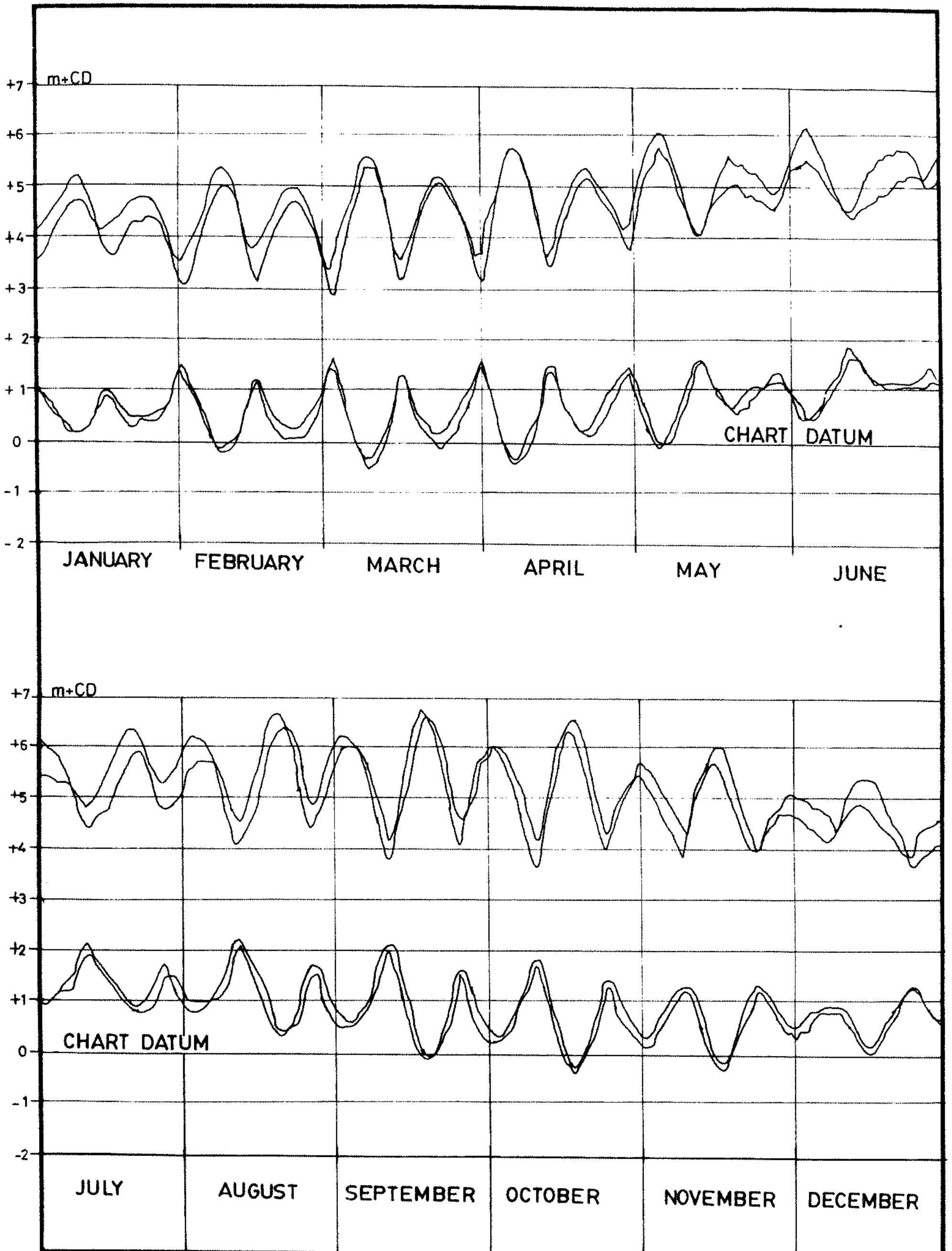
- the daily inequality of the tides
- the fortnightly variations and
- the seasonal variation of the mean level and of the tidal range.

The levels are given above chart datum (CD), which is 1.86 m below Project Datum (PD); for details on Project Datum see also Subsection C.2.2.

The predicted extreme water-levels and tidal ranges at neap and spring tides are collected in Table C.1. Low water-levels vary between PD - 0.5 m and + 2.2 m and high waters between PD + 2.9 m and + 6.7 m. The water-level differences of the neap tide varies between 1.3 m and 3.8 m and of the spring tides between 4.1 m and 6.8 m. The lowest tidal ranges for spring tides occur during December, January and February and the highest in August, September and October.

Predictions for the local stations at Char Pir Baksh and Sandwip North are not available. To obtain usable estimate, the HW and LW observed at these stations during various periods have been correlated to observed HW and LW at Satal Khal (see Section C.2.4).

Owing to shallow water effects, reflection and bed friction, the tidal wave is distorted. Mainly as a result of differences in depth between Hatia and Sandwip channels, the distortion is not equal in these channels, which leads to phase and water-level differences at "the ends" of the channels. The tidal wave through Sandwip channel reaches the secondary channels between Noakhali mainland and Sandwip island somewhat earlier than the wave through Hatia channel. As a result of the differences mentioned above the tidal meeting point is not fixed, but moves over a rather long distance during the tidal cycle.



PREDICTED TIDE AT SATAL KHAL (1985)

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG C.2

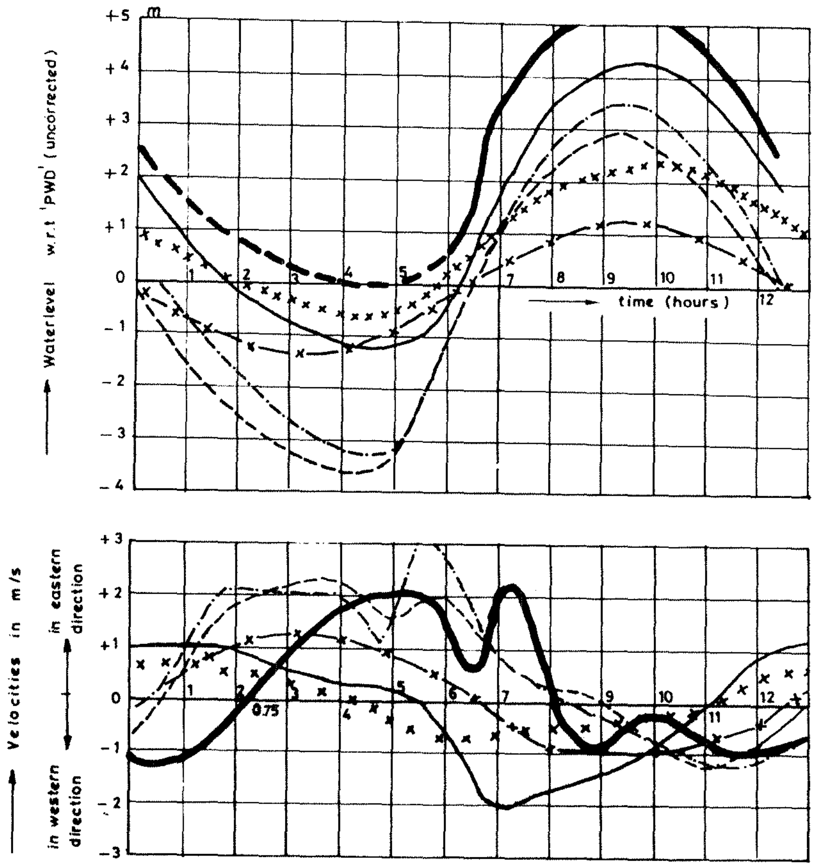
Table C.1 - Predicted LW, HW and range for neap and spring tide at Satal Khal during 1985

Month	Neap tide				Spring tide			
	Day	HW	LW	Range	Day	HW	LW	Range
January	1	3.6	1.4	2.2	11	5.2	0.2	5.0
	17	3.7	1.0	2.7	25	4.8	0.4	4.4
February	1	3.1	1.4	1.7	8	5.4	-0.2	5.6
	15	3.2	1.2	2.0	23	5.0	0.1	4.9
March	2	2.9	1.6	1.3	9	5.6	-0.5	6.1
	15	3.2	1.3	1.9	23	5.2	-0.1	5.3
	31	3.2	1.6	1.6	7	5.8	-0.4	6.2
April	14	3.5	1.5	2.0	21	5.2	0.2	5.0
	29	3.8	1.5	2.3	5	6.1	-0.1	6.2
May	13	4.1	1.6	2.5	20	5.1	0.6	4.5
	28	4.6	1.4	3.2	3	6.2	0.5	5.7
June	11	4.5	1.9	2.6	21	5.2	1.1	4.1
	27	5.3	1.5	3.8	3	6.1	0.9	5.2
July	12	4.4	2.1	2.3	22	6.3	0.8	5.5
	27	4.8	1.7	3.1	2	6.2	0.8	5.4
August	11	4.1	2.2	1.9	19	6.6	0.3	6.3
	25	4.4	1.7	2.7	1	6.2	0.5	5.7
September	9	3.8	2.1	1.7	16	6.7	-0.1	6.8
	23	4.1	1.6	2.5	30	6.0	0.2	5.8
October	8	3.7	1.8	1.9	16	6.5	-0.3	6.8
	22	4.0	1.4	2.6	29	5.7	0.3	5.4
November	6	3.9	1.3	2.6	14	6.0	-0.2	6.2
	20	4.0	1.3	2.7	28	5.1	0.5	4.6
December	5	4.2	0.9	3.3	13	5.4	0.1	5.3
	21	3.9	1.3	2.6	31	4.9	0.6	4.3

Levels are relative to Chart Datum (CD), which is 1.86 m below Project Datum (PD). Underlined figures are highest and lowest values.

In the channel north-west of Sandwip there is a current from Sandwip channel to Hatia channel around local HW and a current in the other direction around local LW (Figure C.3). The current velocities in the deep parts of the channels are rather high, in particular around local LW. Under spring tide conditions maximum velocities in the largest secondary channels have been measured of up to 2.0 to 2.5 m/s and occasionally even up to 4.0 m/s during short periods. A similar pattern is found in the western channel.

The flow situation displayed in Figure C.3 is subject to considerable variation, mainly depending on differential distortion of the tidal waves on a particular day. The current direction may (but need not) reverse several times around the "real" slack water moment. Continuation of measurements during a prolonged time (say two weeks) is required to gain a better insight in this phenomenon.



——— currents during spring tide 9.3.85 in channel between Char Pir Baksh and Char Balua

------ currents during spring tide 9.3.85 in channel between Char Pir Baksh and Sandwip

—·—·— currents during spring tide 30.7.84 in channel between Char Pir Baksh and Sandwip

xxxxxxx currents during neap tide 16.3.85 in channel between Char Pir Baksh and Char Balua

—x—x—x currents during neap tide 16.3.85 in channel between Char Pir Baksh and Sandwip

——— currents during spring tide 15.7.84 in channel between Char Pir Baksh and Char Lakhi

FLOW CONDITIONS IN THE CHANNELS WEST AND EAST OF CHAR PIR BAKSH

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.3

C.2.2 Project Datum

For the purpose of preparation of a mathematical model the reduction of all soundings to one horizontal plane is required. In principle the widely used Public Works Datum (PWD) can be used for this purpose. Unfortunately, the PWD datum levels on the islands and the mainland are poorly interconnected.

In January 1985, recommendations for height transfer in the Deltaic region of Bangladesh were made by the Survey Department of the Dutch Ministry of Transport and Public Works. A very accurate height transfer, using one of the recommended methods would be very expensive, while the accuracy thus obtained is higher than necessary for the purpose of (pre-)designing the cross-dam. Therefore a simpler and cheaper method has been used at this stage to connect the local levels of Sandwip and Char Pir Baksh with those on the mainland, which is described below.

A Project Datum (PD) has been defined to which all elevations and water-levels were to be reduced. Similar to the theoretical PWD, PD was chosen at 1.509 ft (= 0.46 m) below GTS (equivalent to SOB), which is the mean sea-level in the ocean. (GTS = Grid Triangular Survey; SOB = Survey of Bangladesh.)

The damping of the tidal wave by friction leads to conversion of a part of its kinetic energy into potential energy and consequently to a gradual rise of the mean water-level along the path of the wave. The effect appears to be proportional to the tidal range and gives rise to a considerable tidal setup which varies with the spring-neap cycle of two weeks. In the usual tidal analysis this phenomenon shows up as the moon-sun fortnightly component (MSF). Assuming a linear relationship between the tidal range and the fortnightly variation, the Mean Tidal Setup (MTS) can be derived from the tidal constants as:

$$MTS = (M2/S2) \times MSF = 2.25\ MSF,$$

in which M2 and S2 are basic tidal constants.

For each station for which the PD-level had to be established, the tidal set-up was calculated and the local mean sea-level was determined from tidal observations during a fortnightly period. The PD-level was found by correction of the local mean sea-level with the tidal set-up, together with averaged corrections for long-term influences of wind direction, salinity and river discharges for the period concerned.

The relations thus established between local PWD levels and PD are as follows:

- Sandwip-north : PD = PWD - 0.84 m
- Char Lakhi and Char Pir Baksh: PD = PWD + 0.20 m.

C.2.3 Simulation of the effects of closures with mathematical models

C.2.3.1 One-dimensional models

Design of the model

In a one-dimensional model of an estuary, the geometry of the area is characterised by a network of channels which connect storage basins. The flows in the channels and the water-levels in the storage areas are calculated in successive time steps. With this method the unsteady flows in the whole system are simulated. The main purposes of the model in this feasibility study are:

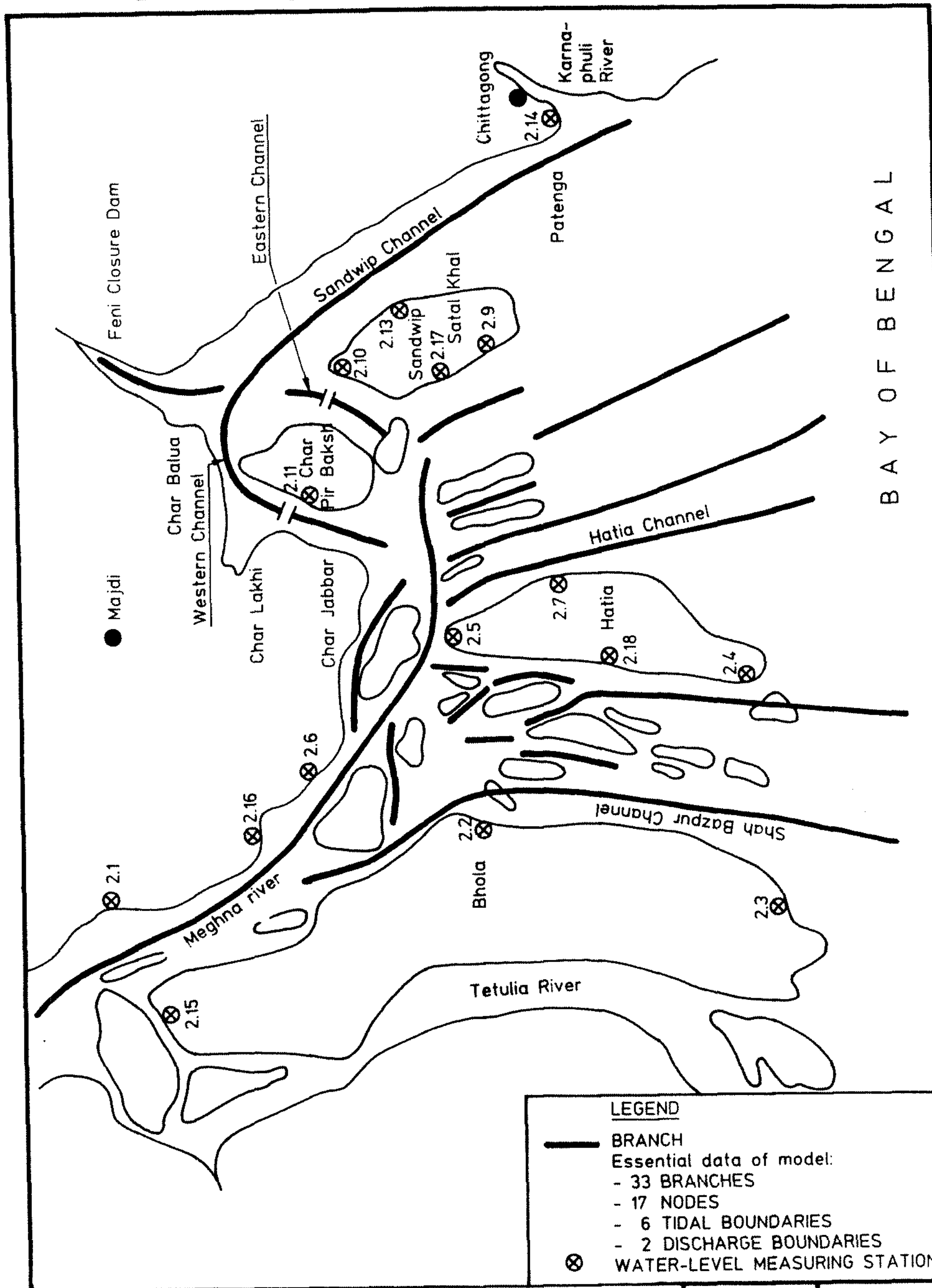
- calculation of the hydraulic phenomena in the present situation;
- calculation of the effects of the proposed works on these phenomena;
- derivation of criteria for the design and execution of the works;
- determination of boundary conditions for more detailed mathematical models.

The model is based on the computer program NETFLOW; a description of this program is given in Appendix C.I.

For the operation of the model it is convenient to have boundaries which can be described as single time curves. (Either a curve with describes the water-level as a function of time, or a curve with describes the discharges as a function of time, but not both at the same boundary). The seaward boundary of the model area roughly follows the depth contour of 10 metres (rel. to CD), running almost WSW from the mouth of the Karnaphuli river as shown in Figure C.4. The northern boundary has been fixed in the Meghna River at the approximate tidal limit in order to allow the northern boundary condition to consist of the river discharge only. Although the actual tidal limit is located much more upstream, the location chosen for the model is adequate for the purpose of designing the Sandwip cross-dam.

The geometry of the area has been modelled in accordance with the most recent sounding maps available. Soundings have been made of the Sandwip channel, the area around Char Pir Baksh (from Sandwip to the mainland), the area between Hatia and the mainland, and the western side of the Lower Meghna river during the pre-monsoon season in 1985. Older (1979-1982) data were used for other parts of the model; for the southernmost part only hydrographic charts were available. As far as possible, the soundings were related to the PD. The rapid morphological development of the area manifested itself in some discrepancies between maps of different years. These discrepancies were "smoothed out" in the model.

An outline of the channels defined in the model is also given in Figure C.4. The channels were chosen to follow the main paths of the flows. The computer program subdivides the channels into sections of 3 km. For each of these, a water-level dependent cross-section is defined. The storage for each module between sections is also calculated



BRANCHES IN ONE DIMENSIONAL MODEL
(NETFLOW & SEFLOW)

Scale 1 : 665,000 (approx.)

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG C.4

as a function of elevation. The time step of the computing process is 15 minutes.

Based on experience in Bangladesh as well as in estuaries elsewhere, a Chézy coefficient of 80 $m^{\frac{1}{2}}/s$ was chosen for the estuarine area. The schematized river has a Chézy coefficient of 40 $m^{\frac{1}{2}}/s$.

The tidal records of the most seaward tidal stations (numbers 2.3, 2.4 and 2.14 in Figure C.4) were used to estimate the conditions along the seaward boundary of the model. The travel duration of the tidal wave was taken into account.

For each relevant season, a neap tide and a spring tide were chosen from the tidal records for reproduction in the model. The tidal curve of one tidal day was made cyclical through harmonic analysis and recomposition.

Based on these considerations, the tide at Patenga was chosen as a reference. The tides at the six boundaries were then defined with an amplitude ratio and a phase shift. Appropriate values of these were obtained by trial and error during the calibration of the model.

Calibration

For the calibration and verification of the model only data from the pre-monsoon observations of water-levels and flows were available. The observed water-levels on 22 and 23 March, 1985 were used for the calibration. No simultaneous flow measurements were available; spring tide observations from the same season were used with correction for the tidal range. During the calibration, adjustments were made with respect to:

- the water depths in poorly surveyed shallow areas (partly using aerial photographs);
- the range and phase of the seaward boundary conditions, and
- the hydraulic friction in the schematic river,

until a satisfactory agreement with the observations was achieved, particularly in the project area.

Performance of the computations for the closure operations

The first computations were performed at the Delft Hydraulics Laboratory in the Netherlands on a Cyber 125 computer using the NETFLOW program. A version of this program is also available for micro-computers under the name SEFLO. This program was used for an identical model on a GRID Computer (512 K), which has recently been added to the computer facilities of the Land Reclamation Project.

Calculations with the NETFLOW model

The spring tide used has a tidal range of 4.70 m at the station Satal Khal (Sandwip), which is representative for spring tides around March. The following pre-closure, intermediate and post-closure situations were simulated:

T_o	original situation
T_1, T_2	horizontal closure stages in the western channel
T_3	western channel closed, eastern channel open
T_4, T_5	horizontal closure stages in the eastern channel
T_6	western and eastern channel closed
$T_7..T_{11}$	combinations of horizontal and vertical closures
T_{12}	eastern channel closed, western channel open

The results of the computer runs have been used in the design of the closures and bed-protection works. Neap tides were also simulated for investigations of the various possible closure methods.

Calculations with the SEFLO model

Apart from the tide with a tidal range of 4.70 m, a tide with a range of 5.60 m, being representative for spring tide in the post-monsoon season, at Satal Khal has been used as well. The following situations were simulated:

Western channel

Closure-gap width 1500 m

S_{1A}	no sill
S_{16A}	sill crest at PD - 2.0 m
S_{2A}	sill crest at PD - 1.5 m
S_{3A}	sill crest at PD + 0.0 m
S_{4A}	sill crest at PD + 1.0 m
S_{15A}	sill crest at PD + 2.0 m

Closure-gap width 1000 m

S_{2B}	sill crest at PD - 1.5 m
S_{3B}	sill crest at PD + 0.0 m

Closed western channel

S_6	eastern channel open
S_{13}	eastern channel closed

Eastern channel

Closure-gap width 4000 m

S_{8A}	sill at PD - 3.0 m

Closure-gap width 2800 m

S_{6B} no sill
S_{7B} sill crest at PD - 5.0 m
S_{8B} sill crest at PD - 3.0 m
S_{9B} sill crest at PD - 1.0 m
S_{10B} sill crest at PD + 0.0 m

Closure-gap width 2100 m

S_{8C} sill at PD - 3.0 m
S_{9C} sill at PD - 1.0 m
S_{10C} sill at PD + 0.0 m

Closure eastern channel

S_{13} western channel closed

The results of the above runs have been used in the design of the closures and bed protection works.

In Figure C.5 an overall picture is presented of the maximum velocities during construction for a vertical closure with a closure-gap width of 1500 m in the western channel and 2800 m in the eastern channel. The tide used has a range of 4.70 m. (For a higher tidal range the velocities are higher, but such tides would occur in a season which should preferably not be used for the final closure works).

The model has also been used for the study of cyclone phenomena by introducing a surge height on the seaward boundary, together with an internal wind effect on the branches.

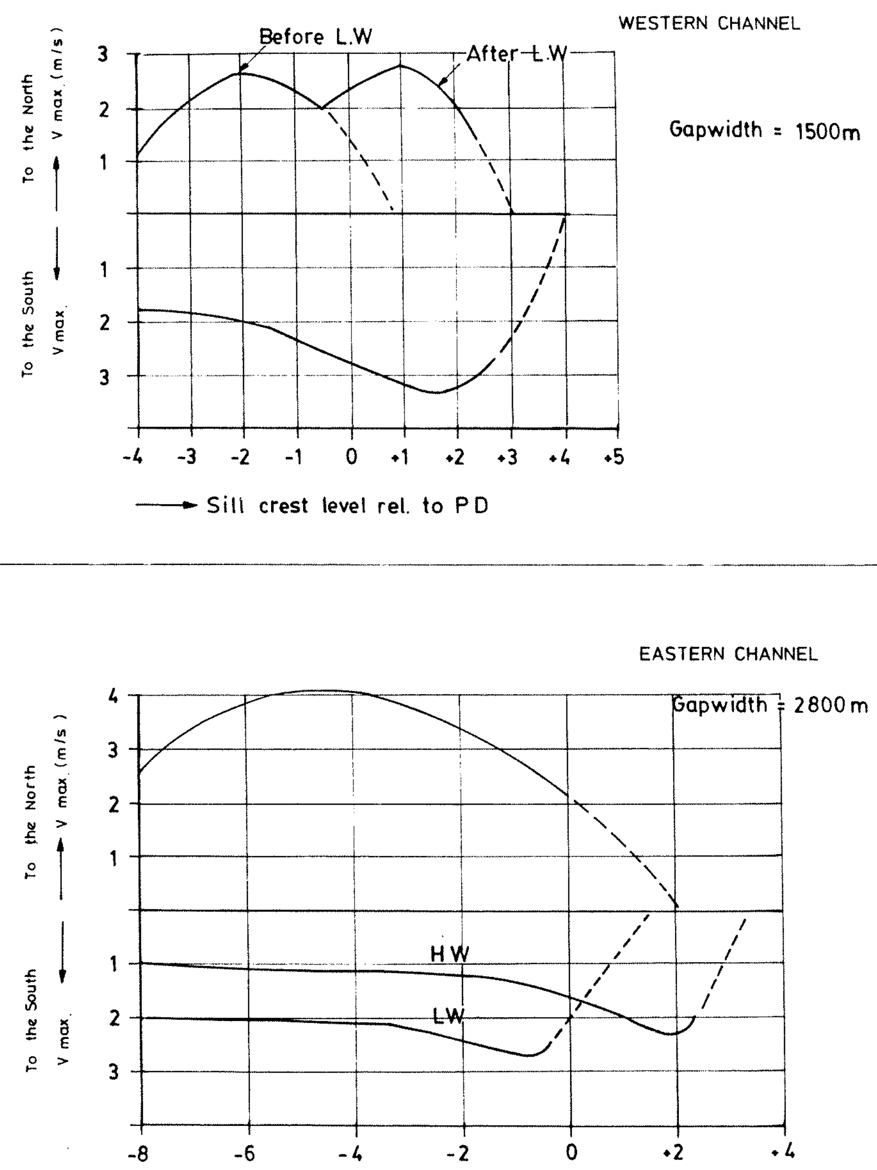
C.2.3.2 Two-dimensional models

The two-dimensional modelling has been based on the computer program WAQUA (recently renamed to DELFLO/DELQUA). This program enables two-dimensional computations of the water movement. It was originally developed by the Rand Corporation (USA) and has been redesigned by the Data Processing Division of the Netherlands Department of Transport and Public Works and the Delft Hydraulics Laboratory. The program provides a numerical solution of the vertically integrated two-dimensional long wave equations. A description of the WAQUA program is presented in Appendix C.II.

The purpose of the two-dimensional models is to provide flow patterns:

- before and after closing of the gaps; this information is used for prediction of the accretion after the closures have been completed.

- during closure stages; this information is used for designing various closure structures.



MAXIMUM VELOCITIES DURING CONSTRUCTION FOR VERTICAL CLOSURES

SPRINGTIDE 4.70 m

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG C.5

Use was made of an overall model and two detail models, as described below.

The overall model (grid size 1100 m)

The overall model comprises roughly the eastern part of the original overall model which was used in the pre-feasibility study (LRP, technical report no. 19, December 1984). The western boundary of the present model runs from Hatia North to Char Jabbar on the mainland (see Figure C.4 for locations). The western part of the original model has not been considered here, as the effects of the dams on this area can be neglected. The bed topography used corresponds with the bed topography in the original overall model, except for those locations where more recent bathymetric surveys were made.

The detail models

Two detail models have been set up (see Figure C.6)

- the western model, comprising the channel between Char Pir Baksh and the mainland; grid size: 250 m.
- the eastern model, comprising the channel between Char Pir Baksh and Sandwip; grid size: 350 m.

The topography in the detail models follows the overall model as closely as possible. The bed roughness distribution in the detail models corresponds to the roughness values used in the overall model.

At the open boundaries, the tidal motions computed with the overall model have been imposed. The orientation of the computational grid allows the closure structures to coincide with a grid line.

The following computer runs were made with the detail models:

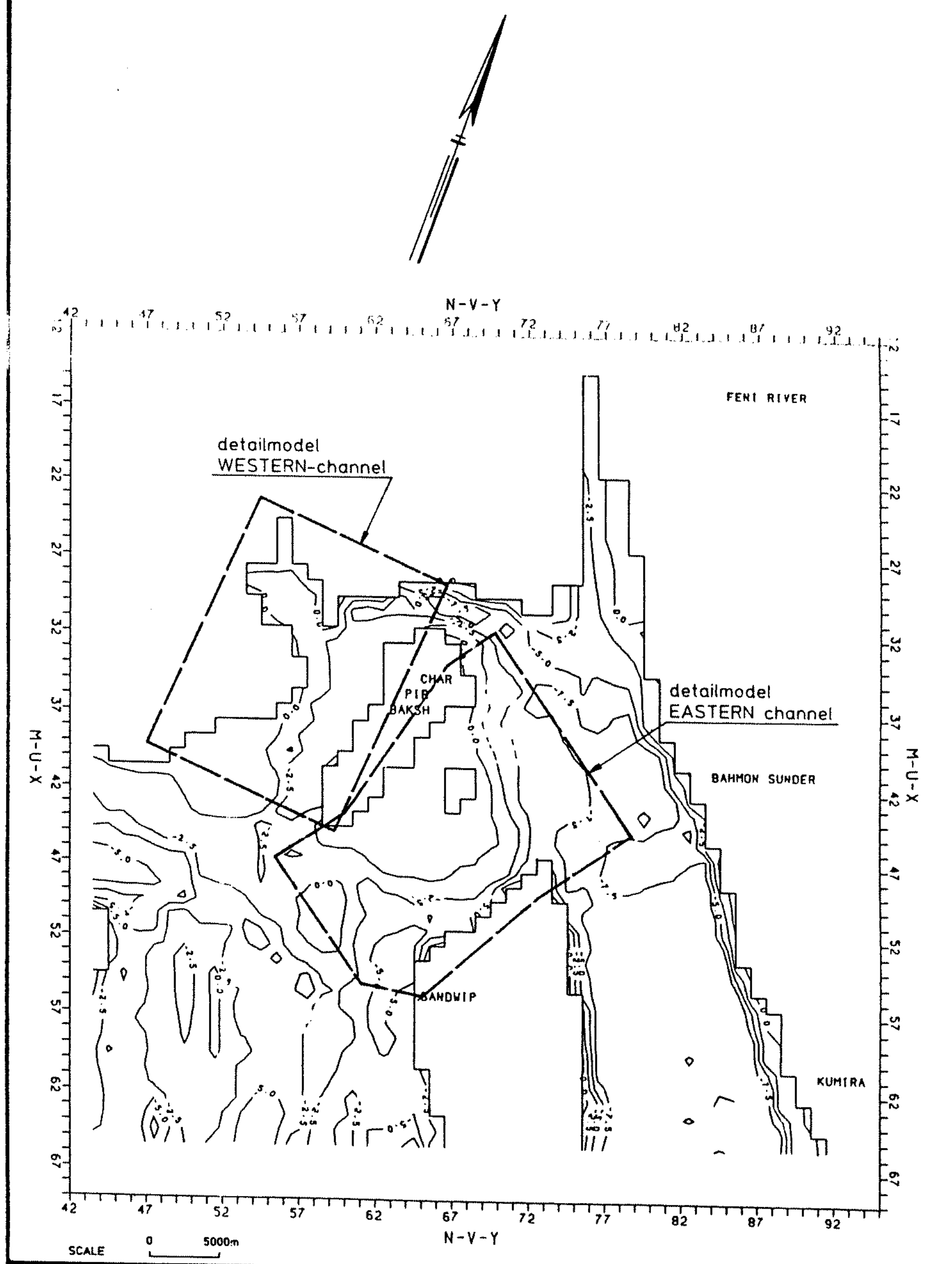
Western detail model

S_0	original situation
S_{1A}	closure-gap width 1500 m, without sill
S_{2A}	closure-gap width 1500 m, sill crest at PD - 1.5 m
S_6	west gap fully closed

Eastern detail model

S_0	original situation
S_8	closure-gap width 5600 m, sill crest at PD - 3.0 m
S_{12A}	closure-gap width 2800 m, without sill
S_{12B}	closure-gap width 2800 m, sill crest at PD - 3.0 m
S_{13}	west and east gap fully closed

S_0, S_6 and S_{13} have been used in the investigations on morphological effects. The other runs have been used in the design of the closures.



DETAIL MODELS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.6

C.2.4 Water-levels, wind and wind setup

Water-level

For a long period water-level data have been collected near Satal Khal at the western side of Sandwip. The predicted (astronomic) water-levels of this station are published on a yearly basis by BIWTA. Predicted and observed water-levels are well related. For the predicted HW- and LW-levels at Satal Khal frequency curves have been drawn based on data from 1981 to 1985 (Figure C.7), both for the monsoon and the winter period.

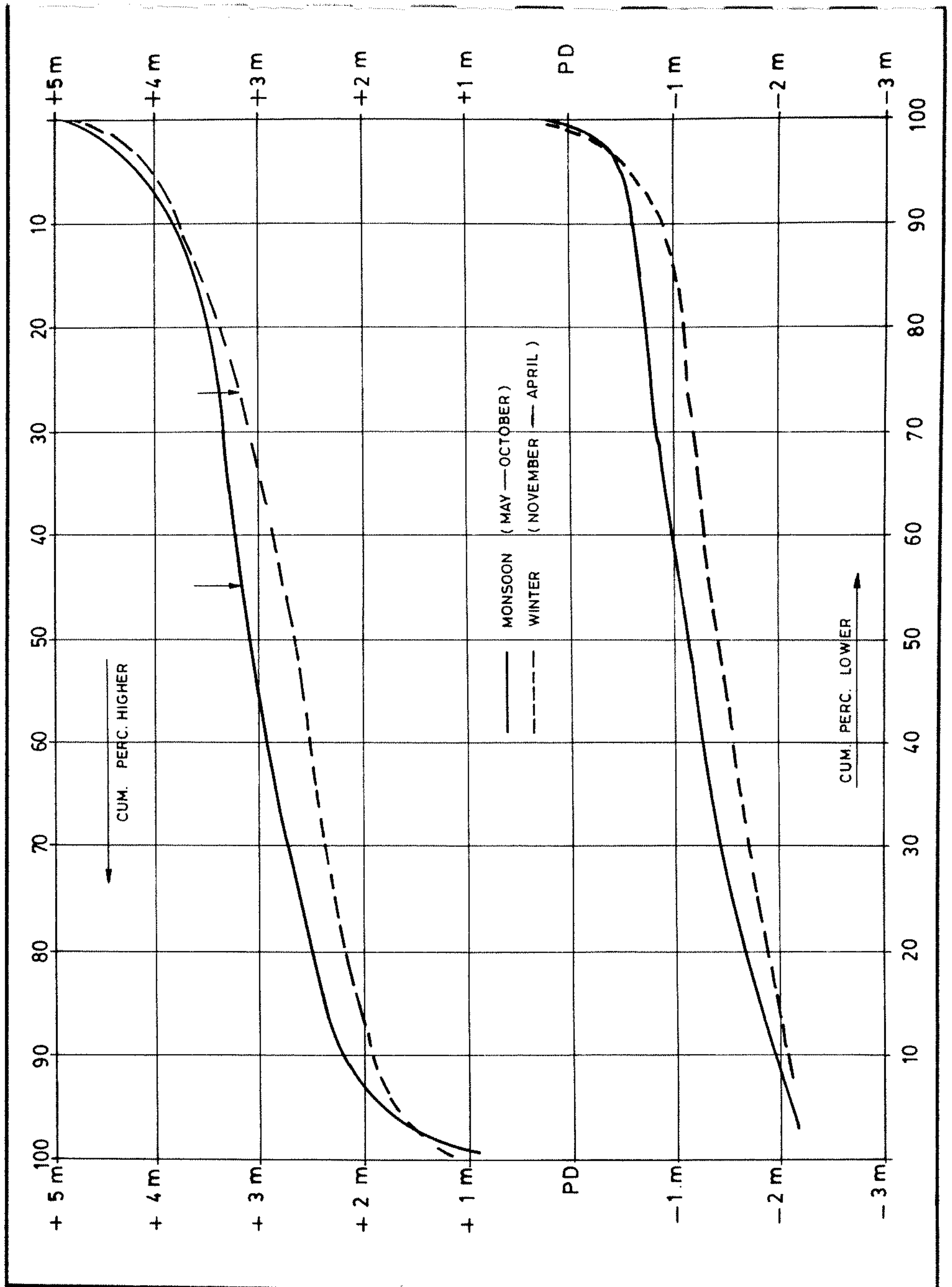
For the design of the Sandwip cross-dam it is necessary to have data for the water-levels in the two main channels to be crossed. Two situations are important: the present situation (T_0), and the final situation (T_{13}) when the channels have been closed. For both situations the expected (astronomical) tidal water-levels have been determined with the SEFLOW model, both for the period December to January (when the average spring-tidal range is 4.70 m) and for the period October to November (when the average spring-tidal range is 5.60 m). This has been done for:

- Satal Khal;
- Sandwip north (the eastern channel);
- Char Pir Baksh (the western channel).

Figure C.8 shows the relation between the astronomic HW-levels at Satal Khal and at Sandwip north (measuring stations 2.17 and 2.10 in Figure C.4). Figure C.9 shows this relation for Satal Khal and Char Pir Baksh. From these figures it can be concluded that in the present situation there is little difference between the HW-levels at Satal Khal and those in the eastern channel. The HW-levels in the western channel are approximately 0.60 m higher than the HW-levels at Satal Khal.

In the final situation (after closure) the HW-levels in the eastern channel will be about the same as in the present situation; the water-levels at both sides of the dam are almost equal.

In the western channel the HW-levels at the north side of the dam will be approx. 0.40 m higher than in the present situation (and more for extreme high waters), whereas the HW-level at the south side of the dam will be approx. 0.50 m lower. Table C.2 shows the relations between the astronomic HW water-levels at various locations:



FREQUENCY LINES FOR ASTRONOMICAL HW AND LW AT SATAL KHAL (1981—1985)

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.7

SANDWIP CROSS-DAM FEASIBILITY STUDY

RELATION BETWEEN ASTRONOMIC HW LEVELS AT SATAL KHAL AND THE EASTERN CHANNEL

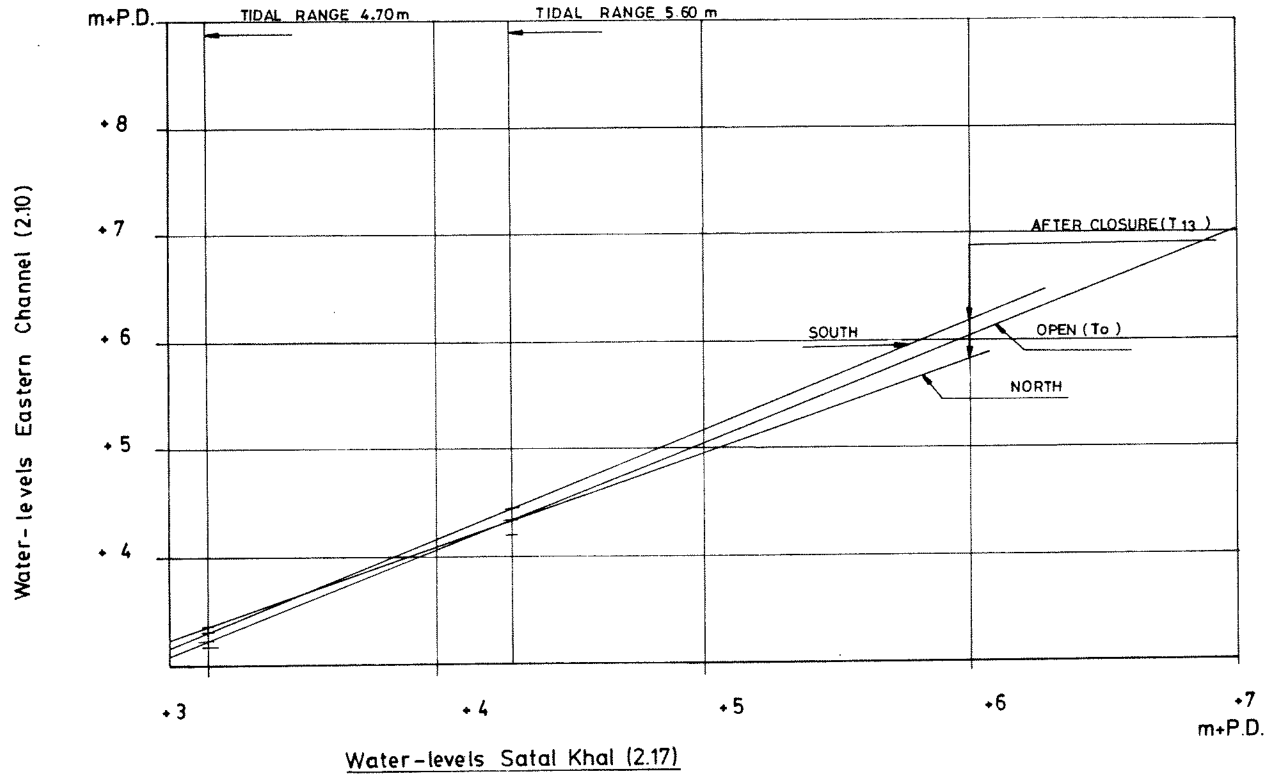


FIG. C.8

SANDWIP CROSS-DAM FEASIBILITY STUDY

RELATION BETWEEN ASTRONOMIC HW LEVELS AT SATAL KHAL AND THE WESTERN CHANNEL

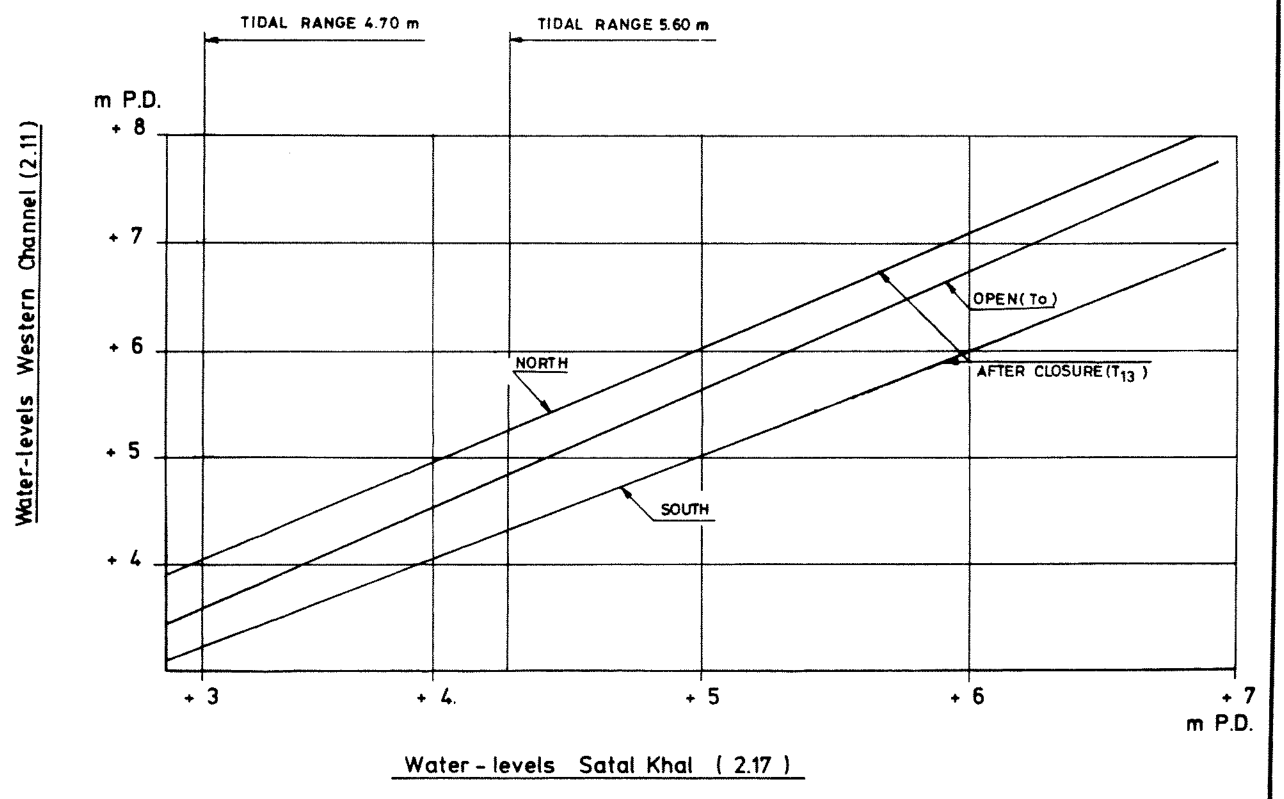


FIG. C.9

Table C.2 - Relations between astronomic water-levels

HW levels in m above PD							Frequency of exceedance (in % of high waters
Satal Khal	Eastern channel			Western channel			
	Present situation	Final situation		Present situation	Final situation		
		North	South		North	South	
2.25	2.35	2.55	2.40	2.60	3.10	2.35	90 %
3.20	3.25	3.35	3.30	3.65	4.10	3.25	50 %
3.80	3.75	3.85	3.95	4.30	4.75	3.85	10 %
4.75	4.80	4.75	4.90	5.35	5.95	5.00	1 %

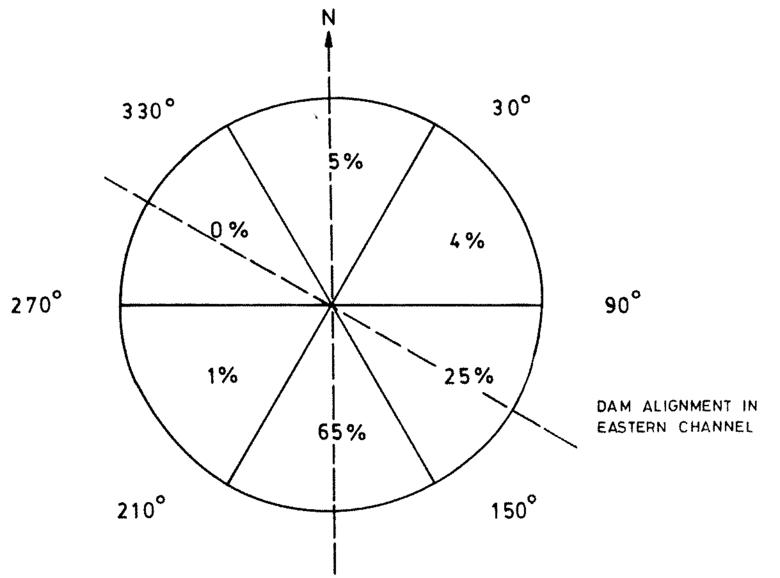
Wind

Wind (including cyclones) also influences (high and low) water-levels. The wind effect depends on velocity, duration and direction of the wind.

Figure C.10 shows the distribution of the wind directions as a percentage of the time that the windspeed exceeds 5 m/s. Figure C.11 shows the number of occasions that the windspeed exceeds certain values.

From Figure C.10 it can be concluded that the dominant wind direction is between south-west and east. Fig C.11 has been based on data of the years 1981 - 1985, for which period the average wind speeds in hours/year are shown below.

Average wind speed in m/s	5	7.5	10	12.5	15	17.5	20	25	30	35	40
Time in hours/year	19	120	100	36	10	4	1	-	-	-	0.6



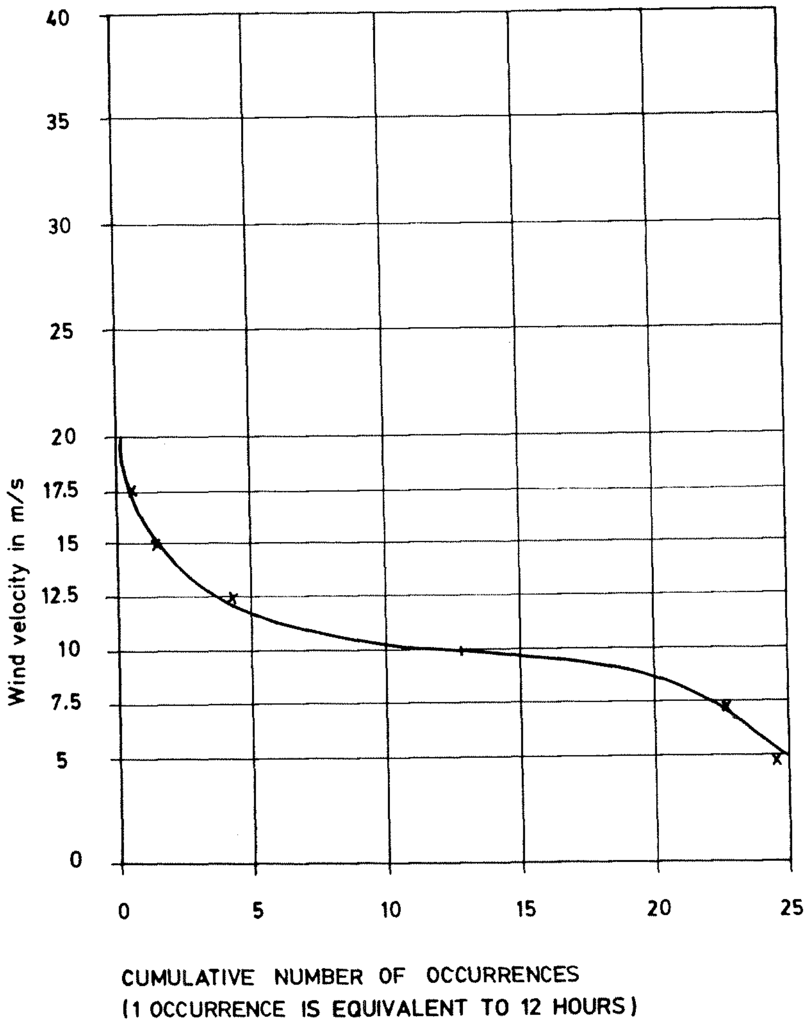
NOTE–
ONLY THE DURATION OF THE WIND DIRECTIONS CORRESPONDING TO WIND VELOCITIES OF MORE THAN 10 KNOTS (5 m/s) HAVE BEEN TAKEN INTO ACCOUNT.

SECTION OF THE COMPASSCARD	DURATION OF WINDSPEED MORE THAN 10 KNOTS	
DEGREE	HOURS	%
	1981 to 1985	
330 — 30	69	5
30 — 90	52	4
90 — 150	359	25
150 — 210	957	65
210 — 270	18	1
270 – 330	6	-

DISTRIBUTION OF THE WIND DIRECTIONS IN PERCENTAGES OF TIME

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C10



WIND VELOCITY DISTRIBUTION
AVERAGES 1981 — 1985

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C11

In general the wind speed seldom exceeds 15 m/s (Beaufort number 7). Only in cyclonic circumstances will the wind speed be substantially higher. The value of 40 m/s mentioned in table C.2.3 was measured on Sandwip during the cyclone of May 1985. According to information received from the Meteorological Institute in Dhaka, the wind speed in the period 1970 - 1980 was only once significantly higher than 20 m/s. This occurred during the cyclone of November 1970 when a wind velocity of 138 miles per hour or 61 m/s was measured near Chittagong.

There is little information on the consequences of extremely high wind speeds on the water-levels. According to information collected from various sources the cyclone of May 1985 caused a surge with a height of 2 m near Sandwip and of 3 m near the Feni Closure Dam and Char Pir Baksh. The surge caused by the cyclone of 1970 was approx. 50 % higher.

Based on the information collected so far it is estimated that the wind setup in both channels may reach the following heights:

- 0.5 to 1.0 m for wind velocities in the range of 7.5 to 12.5 m/s
- 1.0 to 1.5 m for wind velocities in the range of 12.5 to 17.5 m/s
- 1.5 to 5 m for higher wind velocities, including severe cyclones.

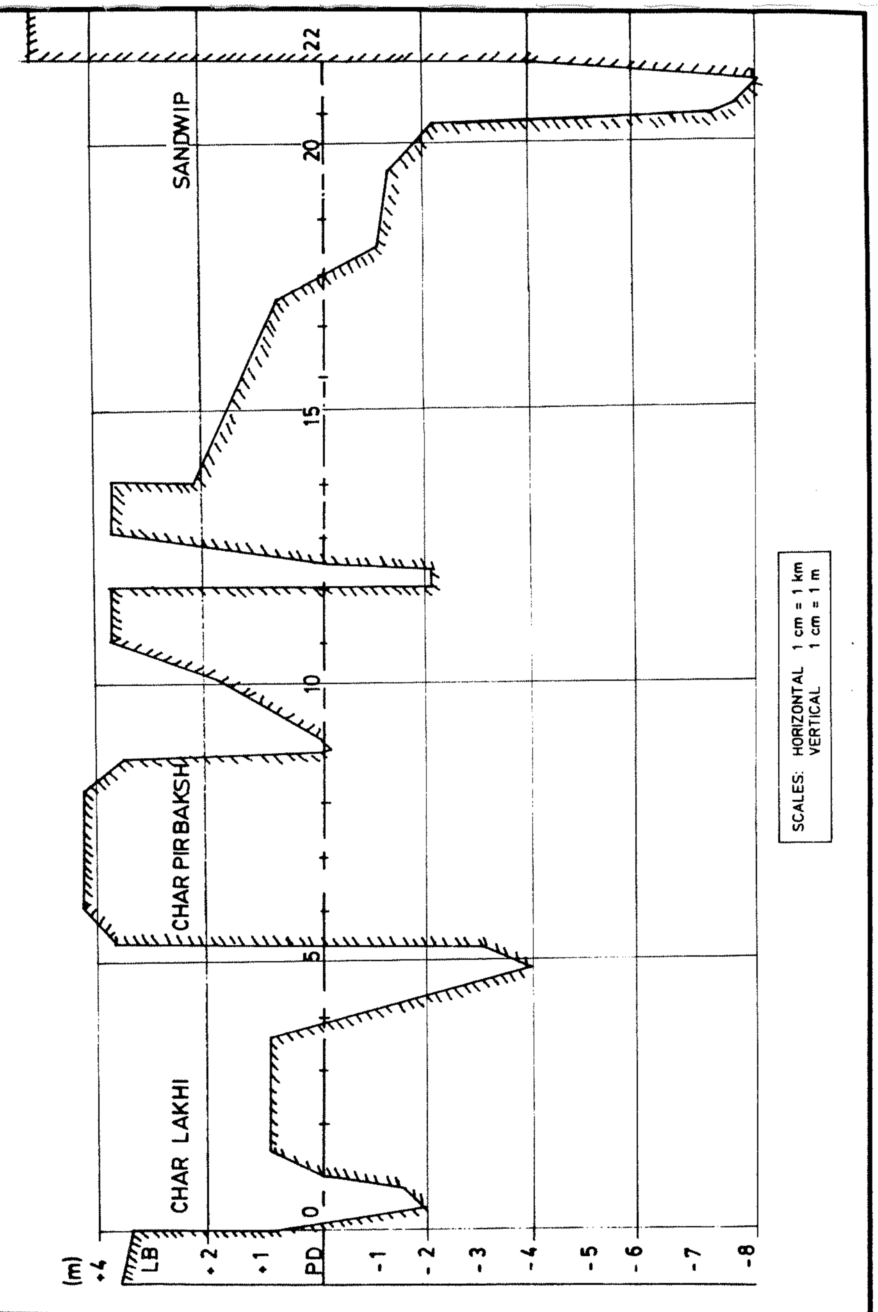
These setup values should be added to the astronomical (high) water-levels to obtain the maximum possible water-levels at the closure sites.

C.2.5 Waves

Data on wind waves, swell and long-period waves are not available for the Sandwip cross-dam area. The Meteorological Institute in Dhaka however possesses information on wind speed, wind direction and wind duration in that area. For different wind directions (sectors) the fetch (the uninterrupted friction length between wind and water) has been determined. This fetch has been taken as the maximum possible length in a certain wind sector.

The data on wind duration and fetch for all wind speeds of more than 10 knots (5 m/s) in the years 1981 - 1985 have been used for estimating wave parameters. This has been done by using a wave-forecasting diagram that gives the empirical relations between a given fetch, duration and wind speed and wave parameters, such as significant wave height (Hs) and wave period (T). These relations apply to deep water.

The wave height in the Sandwip cross-dam area is limited by the water depth. There is a substantial difference between the present sea bed level and the level expected some years (or even one year) after the closures. The sea bed level along the preliminary alignment of the cross-dam is shown in Figure C.12.



PROFILE ALONG PRELIMINARY ALIGNMENT

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.12

Fig C.13 shows the wave heights as a function of the water depths. It can be concluded that shortly after closure (say 1 to 2 years) the maximum wave height is only determined by the water depth.

C.2.6 Geotechnical boundary conditions

C.2.6.1 Seismicity in the proposed dam area

Data on the seismicity in and around the project area were obtained from the Royal Dutch Meteorological Institute KNMI.

The Sandwip cross-dam is situated outside the seismic zone, which reaches from northern India via Burma to Indonesia. In the period 1900-1983 earthquakes of magnitude M (Richter scale) have occurred at the distances from the construction area given in Table C.3.

Table C.3 - Distances of earthquakes from dam site

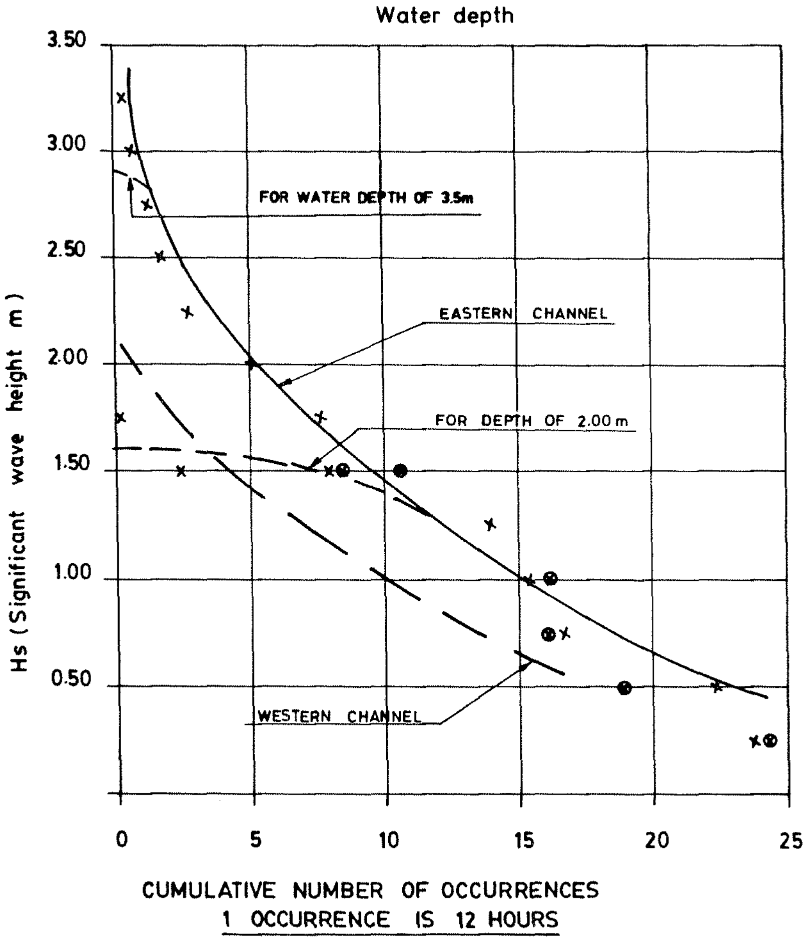
M-class	Distance	Accelerations in epicentre
5 - 5.9	100 km	0.04 - 0.1 g
6 - 6.9	150 km	0.1 - 0.5 g
7 - 7.9	200 km	0.5 - 2 g
8 - 8.9	300 km	2 - 4 g

Source : KNMI

The accelerations at some distance from the epicentre depend on the actual distance. The following accelerations would have occurred at the cross-dam site (see Table C.4).

Table C.4 - Accelerations at the dam site (in g)

M-class	Distance from epicentre			
	100 km	150 km	200 km	300 km
5 - 5.9	0.002-0.005	0	0	0
6 - 6.9	0.005-0.025	0.003-0.015	0	0
7 - 7.9	0.025-0.1	0.015-0.06	0.01-0.04	0
8 - 8.9	0.1 -0.2	0.06 -0.12	0.04-0.08	0.02-0.04



SIGNIFICANT WAVE HEIGHT Hs IN THE WESTERN AND EASTERN CHANNELS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG.C13

The highest accelerations at the site which would have occurred according to Tables C.3 and C.4 were the result of M 7.9 quakes at a distance of 200 km (0.04 g) and of M 8.9 quakes at a distance of 300 km (0.04 g). The weaker quakes at shorter distance would have caused accelerations below 0.015 g (M 6.9 quakes at a distance of 150 km).

If it is assumed that the heaviest earthquakes may occur anywhere within the seismic zone, the maximum acceleration at the site would be 0.2 g. (An M 8.9 quake at a distance of 100 km). The frequency of occurrence of M 8 - 8.9 earthquakes in the total seismic area considered is three times in 90 years. Assuming a random distribution within the seismic zone, the chance of occurrence in an area with a 100 - 150 km radius from the dam location is approx. 1:1000 per year.

However, seismic-tectonic data indicate that the heaviest earthquakes are to be expected mainly in the centre of the seismic zone. This means that the chance of occurrence of an M 8 - 8.9 earthquake near the dam location will in fact be far less than 1:1000 per year.

Because of this small chance, an acceleration of 0.04 g has been used in various aspects of the dam design, being the value that can reasonably be expected. This value is in accordance with recommendations laid down in other codes.

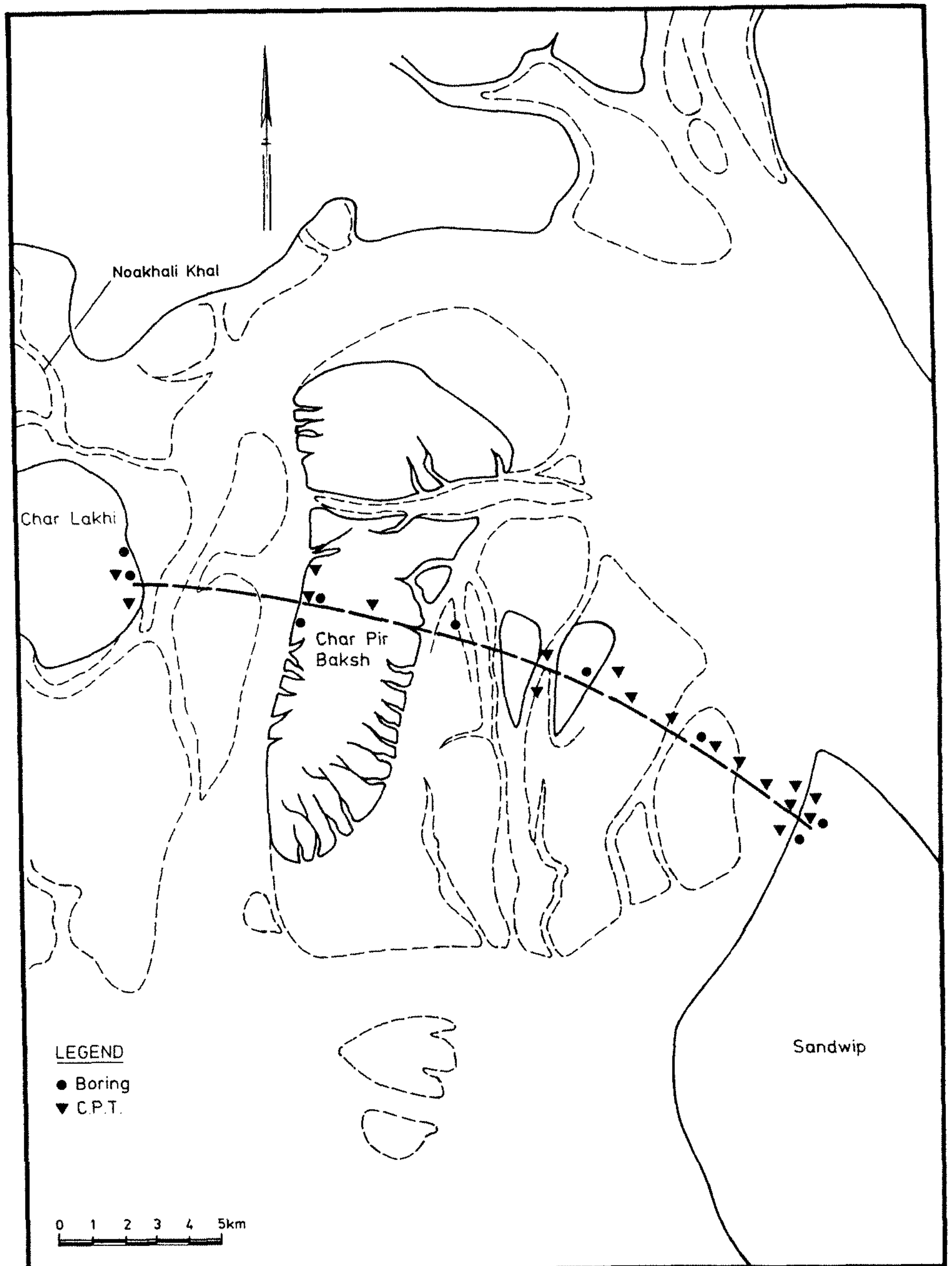
C.2.6.2 Site investigations and laboratory testing

The site investigations were carried out by Soiltech International Limited, Dhaka and comprised 25 Dutch Cone Penetration Tests (CPT) and 9 boreholes (Fig. C.14).

The greater part of the boring and CPT locations was situated in the axis of the alignment of the proposed dam. Some locations were 1 km north and 1 km south of the axis. Approx. 35% of the locations were situated on the shores and shoals, approx. 40% in the shallow water, and approx. 25% in the deeper water. A number of CPT locations coincided with borehole locations in order to determine the relation between CPT values and type of soil.

The boring programme was not carried out completely. Near the end of the period of execution the weather and sea conditions deteriorated and the boring had to be abandoned for safety reasons. Four CPT's out of the planned 29 were not executed. These four locations are situated in the channel between Char Pir Baksh and Char Lakhi. As mentioned in Section C.16, item (m), additional geotechnical investigations in the alignment of the cross-dam need to be carried out in the final design stage, particularly in the western channel but also in the eastern one, to complete the programme.

Laboratory tests on selected samples comprised sieve and hydrometer analyses, consolidation testing, triaxial testing, determination of Atterberg limits, wet and dry density, specific gravity, moisture content and critical density. These tests were carried out by Soiltech International Ltd. and the Soil Mechanics & Material Directorate of the River Research Institute of the Bangladesh Water Development Board (BWDB).



LOCATIONS OF SUBSOIL INVESTIGATIONS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.14

The results of the site investigations and subsequent laboratory testing were reported by Soiltech in their reports of August 1985 and January 1986 and by the Soil Mechanics & Materials Directorate in their report number 169(85). A summary is given in the following paragraphs.

(a) Site investigation results

The ground levels of the locations were established between PWD + 4.97 m (PD + 5.17 m) at Char Pir Baksh and PWD - 9.94 m (PD - 9.10 m) in the gully close to Sandwip island.

The groundwater tables vary between approx. PWD + 4.50 m (PD + 4.70 m) and sea level. These levels vary strongly with the rainfall and the river discharges.

The penetration depth of the CPT's and the borings was 20 m in all locations.

The composition of the subsoil varies from sandy silt in the top 5 metres to silty fine sand or sandy silt in the deeper layers. Between PD - 5 m and -15 m the sand portion dominates. Below PD - 15 m the silt portion dominates. Clay is hardly encountered. Layers of pure sand have not been found.

At many locations the cone resistance measured in the first 3 m below ground/seabed level is 0.1-1.0 MN/m^2. At some locations the sandy silt will be rather soft. Between depths of 3 to 9 m the cone resistances increase to 1-3 MN/m^2. Between depths of 9 and 14 m the cone resistances are 3-5 MN/m^2. In most locations the cone resistances are less than 4 MN/m^2 below a depth of 14 m.

(b) Laboratory testing results

Sieve and hydrometer analyses have been carried out on 119 samples. The sampling rate was approx. 1 sample per 1.5 m of boring length. According to the sieve and hydrometer tests all samples consisted of sand and silt. The average sand percentage was 41.4 and the average silt percentage was 55.8. In 27% of the samples clay was encountered. The average clay percentage of the 119 samples analysed was 1.8%. The average clay content of the samples containing clay was 7%. The average D_{50} of all samples is 0.056 mm. The minimum D_{50} of all samples is 0.010 mm. The maximum D_{50} of all samples is 0.125 mm. The maximum grain size encountered is 0.200 mm.

Atterberg limits determined on 49 cohesive samples showed an average liquid limit of 40.7 % and an average plastic limit of 24.7 %. The results indicate that the clayey sandy silt is of low to medium plasticity.

The wet densities lie between approx. 15 and 20 kN/m^3. The average wet density is 18.5 kN/m^3. The porosities lie between 40 and 55%; the average is 48%.

Nine consolidated drained triaxial tests have been carried out on cohesive samples, resulting in strength parameters of which the minima, maxima and the average are given below. (The minima and maxima do not refer to the same samples).

	Friction angle in degrees	Cohesion in kN/m^2
Minimum	23	3
Maximum	37.5	25
Average	31	9

Triaxial shear tests in consolidated drained condition were carried out to determine the critical density of 8 selected samples. In 7 out of 8 tests the critical density remains below the in situ density, implying a low sensitivity for liquefaction.

Consolidation tests were conducted on 9 undisturbed samples. The average initial void ratio and compression index were 0.86 and 0.106 respectively.

(c) Conclusion

According to the results of the site and laboratory investigations the subsoil in the alignment of the cross-dam to a depth of 20 m can be characterized as low to medium plastic material, mainly consisting of alternating sandy silt and silty sand layers, highly erosive and difficult to compact. Sensitivity to liquefaction is low.

C.2.7 Dynamics of the channel-shoal-island system

The channel-shoal-island system is not stable. At the present there are four distinct (short-cut) channels between the Sandwip and Hatia channels, namely the western channel, two small central channels and the eastern channel. In the past the western and eastern channels have shown a tendency to shift in an easterly direction, thereby "consuming" parts of Sandwip and Char Pir Baksh. Until recently the central channels were comparatively small, contributing little (in terms of volume) to the exchange of water between the Sandwip and Hatia channels.

The central channels appear to be increasing in size, notably in depth. This increased size may result in increased discharges through them, which in turn may lead to further increase of their size.

Initiation (and stoppage) of the above process is governed by water-level differences at both "ends" of the channels and by the sediment transport capacity of the flow through the channels. Ultimately the increased flow through the channels may influence the water-levels

at both ends of the channels. This increased flow may decrease the flow through the eastern channel. As a result its width and or depth may be reduced accordingly.

At this stage, with relatively limited data on hand, it is difficult to forecast whether the possible redistribution of the exchange flow between the Sandwip and Hatia channels would have any effect on the erosion rate of Sandwip; if it has any effect the erosion rate will probably be lower.

Since little can be done at this stage to control this redistribution process, the design of the proposed cross-dam should be fairly flexible, so that it can easily be adapted before or even after construction has started to the changing natural circumstances.

C.2.8 Climate and workability

This section deals with the climate at and near the site, as far as it affects or influences the construction of the cross-dam. Data and considerations on hydrological conditions, including rainfall, surface water and groundwater are dealt with in Annex F.

For land-based operations (excluding excavation work) it is possible to work during the entire year, except on days with exceptional rainfall. A pre-requisite however is the availability of all-weather access and site roads.

Excavation work can in principle be carried out from the middle of November till the middle of May, but rainfall in late March and April often leads to poor accessibility of the excavation areas.

For marine based operations the winter season is best suited, but with adequate water-borne equipment and anchoring facilities the remainder of the year can also be used, except when wind velocities and waves are too high (see also Sections C.2.4 and C.2.5).

C.3 Alignment of the cross-dam

The alignment of the Sandwip cross-dam, and more specifically the locations of the closures in the main channels, should be determined such that the hydraulic separation of the Sandwip and Hatia channels (which will halt the erosion of Sandwip and Char Pir Baksh, and stimulate accretion of land) can be achieved at minimum cost. Side effects of the closures, such as accretion rate, drainage of adjacent mainland, etc., which may be different for various locations of the closures, should also be considered in the selection process.

The principal cost component of a closure dam is the cost of closing the final gap(s). The final gap(s) may in principle be located either on the shoals or in the gully (or gullies). As will be elaborated in Chapter C.7, the final gaps should preferably be located in the gullies for both the western and eastern closure-dams.

The cost of the closure section is mainly governed by:

- the dimensions (width and depth) of the final closure-gap, and
- the degree of difficulty of the closure.

Both factors are strongly interrelated. The dimensions of the final gap depend, among other things, on the geometry of the channels to be crossed. The degree of difficulty of a closure depends to a large extent on the difference between the water-levels at both sides of the (almost) completed closure dam.

In the western channel, tides coming in from Hatia Channel and from Sandwip Channel meet between Char Pir Baksh and Char Lakhi. Such a tidal meeting point is normally the most suitable location for the construction of a cross-dam. This was confirmed by the results of the two-dimensional mathematical computations carried out during the pre-feasibility study (LRP, 1984a). In the first instance, two alternative alignments were considered: one from Char Pir Baksh to the west along the tidal meeting point to Char Lakhi, and one from Char Pir Baksh to the north across a narrower channel to Char Balwa (the so-called northern alignment). The computations (LRP, 1984b) demonstrated that the closure in the northern alignment would have to cope with head differences of over 3.3 m, twice as much as the head differences to be expected in the closure of the proposed alignment. This would mean that the rock/concrete blocks to be used in the northern alignment would have to be twice as large as the blocks for the proposed alignment, and that they would have to be about eight times as heavy. Moreover, the final closure-gap would have to be considerably wider. A preliminary cost estimate indicated that a dam along this northern alignment would cost approximately Tk 1 400 000 000 more than a dam along the proposed alignment, from Char Pir Baksh to Char Lakhi, following the tidal meeting point.

In Annex D, Section D.6.5, the possible consequences for the drainage function of the Noakhali Khal of both alignments are compared, and there the conclusion is reached that there is no reason to expect that a cross-dam along the northern alignment would be more favourable in this respect. Consequently, a starting point for the western dam south of the Noakhali Khal is to be preferred, which is in accordance with the preliminary alignment. Figure C.14 indicates this alignment. The following sections deal with establishing more precisely the starting points of the various dam sections.

C.3.1 Location of the western (closure-)dam

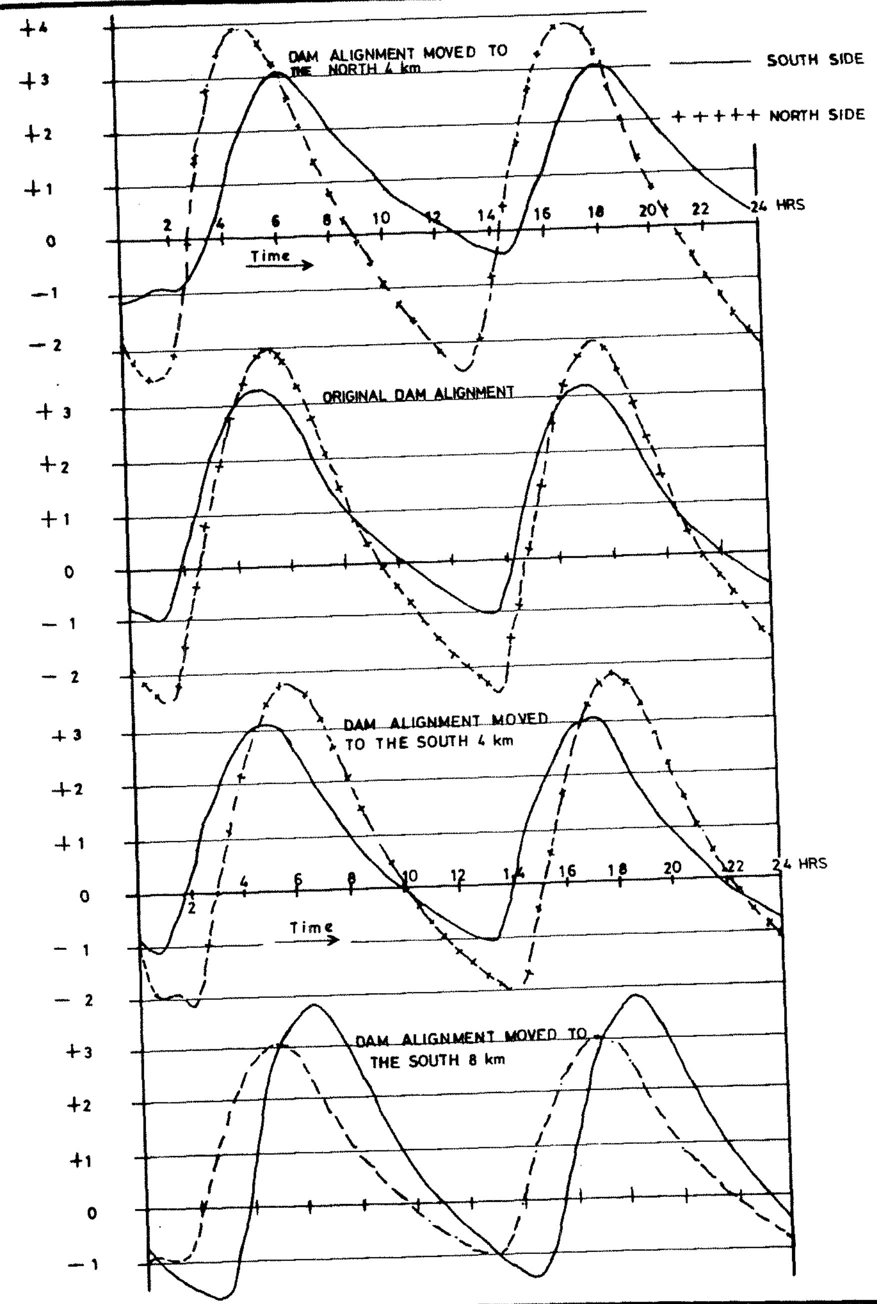
In order to verify whether the preliminary alignment is indeed the most favourable, a number of calculations have been made to determine the maximum head differences in the final closing stages for alternative dam locations (Figure C.15). The main results of these calculations have been assembled in Table C.5.

Figure C.15 further illustrates the water-levels to be expected at both sides of the western dam for the locations investigated. From the table and the figures it can easily be concluded that an alignment coinciding with or near the preliminary alignment is most favourable.

Table C.5 - Maximum head differences for alternative alignments

Alignment	Max. head difference around H.W.	Max. head difference around L.W.
4 km to the North	2.10 m	2.50 m
Prelim. alignment	1.00 m	1.40 m
4 km to the South	1.60 m	2.50 m
8 km to the South	1.80 m	2.70 m

At present a location slightly south (say 1 km) of the preliminary alignment has certain advantages in view of the channel geometry and the high level of the shoals. In view of the rapidly changing situation (possibly cyclic) the final decision of the location of the western dam should be made shortly before the start of the construction.



HEAD DIFFERENCE OVER FINAL CLOSURE GAP FOR DIFFERENT LOCATIONS OF WESTERN DAM

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.15

C.3.2 Location of the eastern (closure-)dam

Hydraulic calculations proved that shifting the location of the eastern closure-dam with respect to the preliminary alignment does not lead to a significant change in the head differences to be expected during the last closure stages. Therefore the (present and future) channel geometry will almost exclusively determine the location of the eastern closure dam.

Soundings made in February 1986 revealed that the deep channel to be crossed near Sandwip had become narrower and deeper. This could imply that the quantity of material needed for building up the sill is somewhat less, compared with earlier estimates. Other considerations, such as flow patterns around the "tip" of Sandwip and current contractions to be expected at the head of the earth dam over the shoals, have led to the conclusion that the width of the final closure-gap should not be less than 2800 m, and that the wet area below PD + 0.0 m should initially not be chosen too small.

From the land accretion point of view a shift in a southern direction of the eastern dam places the dam more towards the centre of the land accretion process. This implies a slightly higher rate of accretion on the average.

Considering the above factors it is recommended that the axis of the eastern closure dam be shifted approximately 1.5 km to the south of the preliminary alignment.

C.3.3 Location of the central closure-dams on Char Pir Baksh

The central closure-dams should ideally be located at or near the tidal meeting point in each channel (i.e. the points where the tidal currents meet so that no current velocity is measured at this point in either direction). At present insufficient information is available to determine the tidal meeting points. For the time being the alignment has been chosen along the connecting line between the western and eastern closure dams. The location of the dam on Char Pir Baksh should be selected in accordance with the most favourable land development plans.

C.4 Sequence of closures

The first step in formulating the closure strategy is to determine the effects of the closure sequence on:

- current velocities and erosion/siltation patterns in the last channel to be closed,
- the closure operations themselves.

For the purpose of this section it has been assumed that the central channels will be closed prior to the western and eastern channel(s). The question then remains which of the latter two channels will be closed first: the western or the eastern channel.

Theoretically it is also possible to close the two channels simultaneously. This involves however the use of additional equipment and more difficult logistics, while none of these closures will become easier. Therefore simultaneous closures have not been considered.

The following situations were simulated in various computer runs with the NETFLOW model (see Section C.2):

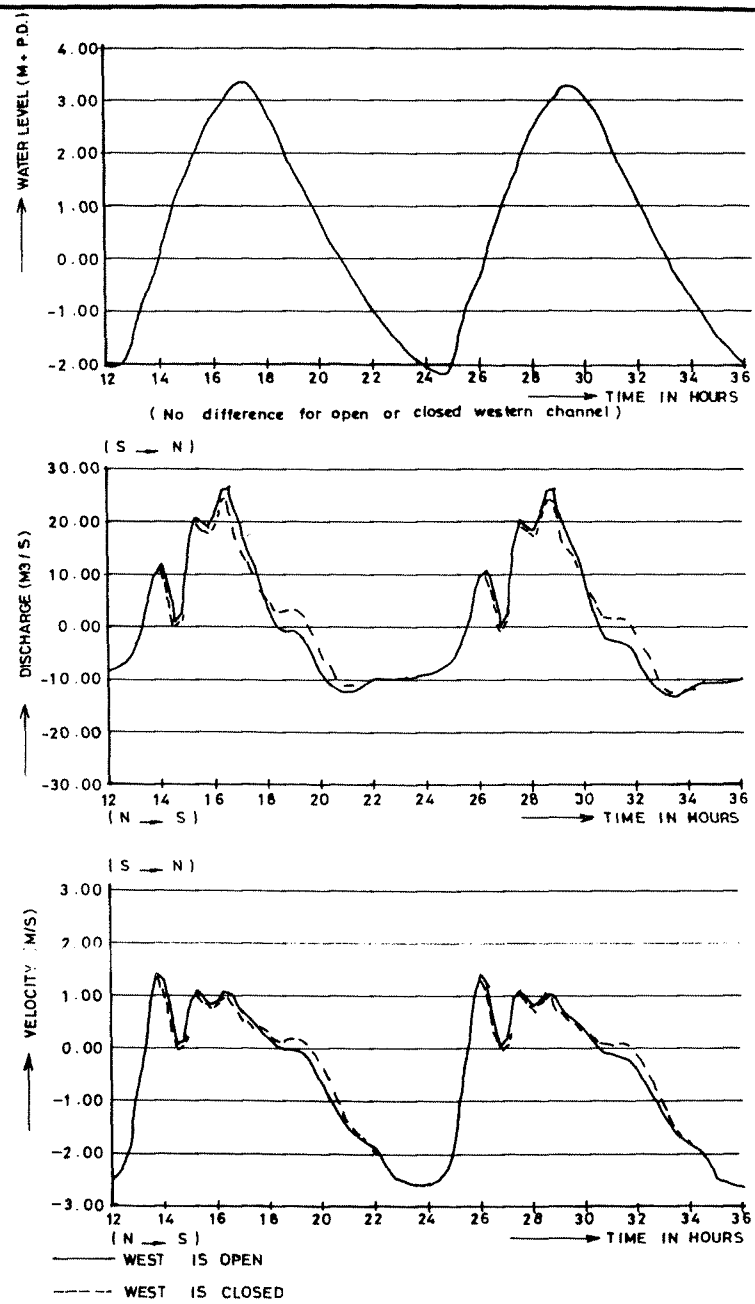
- T_0 : both channels 100% open
- T_3 : West closed, East open
- T_6 : both channels 100% closed
- T_{12} : West open, East closed.

C.4.1 Sequence West - East

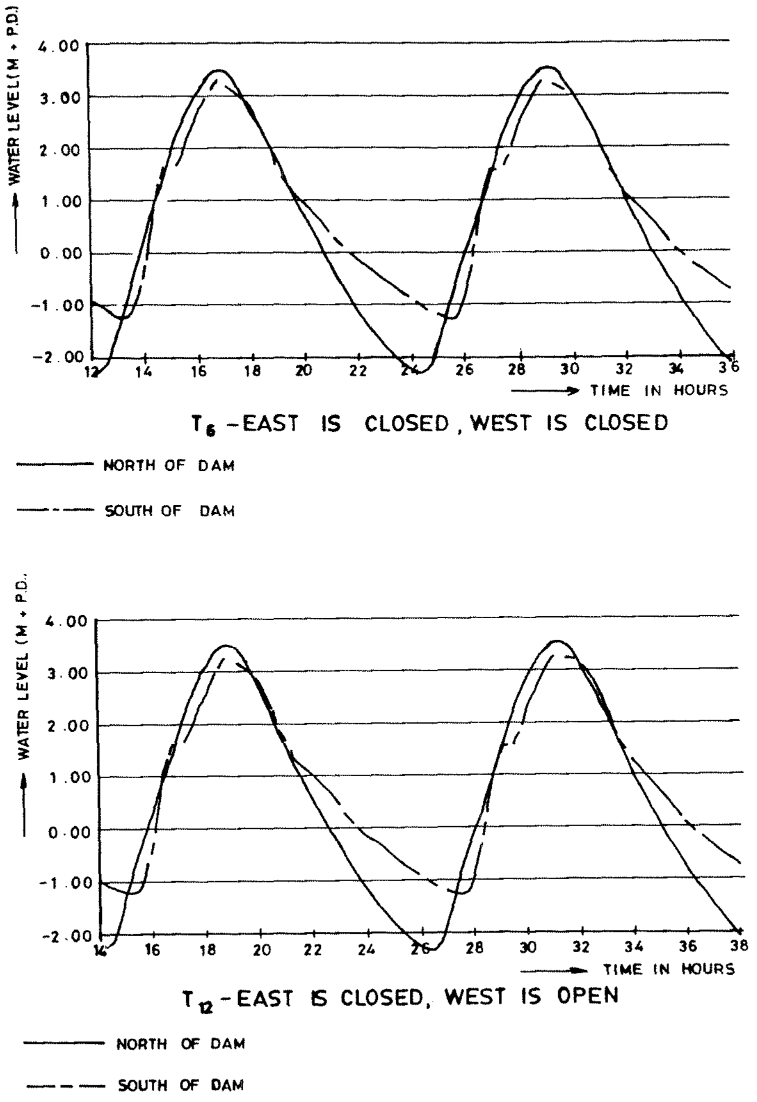
Some of the results of the computer runs with the NETFLOW model (situations T_0 and T_3) for the eastern closure site have been reproduced in Figure C.16. A close scrutiny of this figure leads to the following conclusions, valid for spring tides:

(a) After closure of the western channel the maximum discharges through the eastern channel decrease by 8%; this applies to both flow directions.
(b) The maximum velocities, which do not occur simultaneously with the maximum discharges, will not (or hardly) change.

For the closure of the eastern channel itself it does not matter whether or not the western channel has been closed earlier. This can be concluded from Figure C.17, in which the water-levels at both sides of the eastern dam have been plotted for two situation western channel open (situation T_{12}), and western channel closed (situation T_6).



SEQUENCE OF CLOSURES-WATER LEVELS, DISCHARGES, VELOCITIES IN OPEN EASTERN CHANNEL	T_0/T_3 - EAST
	SPRING TIDE, 4.70m AT SATAL KHAL
SANDWIP CROSS-DAM FEASIBILITY STUDY	FIG.C16



SEQUENCE OF CLOSURES-WATER LEVELS AT BOTH SIDES OF EASTERN DAM AFTER CLOSURE (West open or closed)	T_6/T_{12} - EAST SPRING TIDE, 4.70m AT SATAL KHAL	
SANDWIP CROSS-DAM FEASIBILITY STUDY		FIG.C17

C.4.2 Sequence East - West

Some of the results of the computer runs (situations T_0 and T_{12}) for the western closure site are reproduced in Figure C.18. The following conclusions can be drawn:

(a) After closure of the eastern channel, the maximum discharges in S-N direction in the western channel increase by as much as 35%; the maximum discharges in the other direction do not change significantly.
(b) The maximum velocities in S-N direction, which occur simultaneously with the maximum discharges, also increase by 35%, while the maximum velocities in N-S direction do not change.
(c) The above implies that substantial erosion will occur in the western channel after (the start of) the closure operations of the eastern channel.

The closure operation of the western channel itself would be significantly different depending on whether or not the eastern channel has been closed earlier. If the eastern channel is closed first, the effects in the western channel would be:

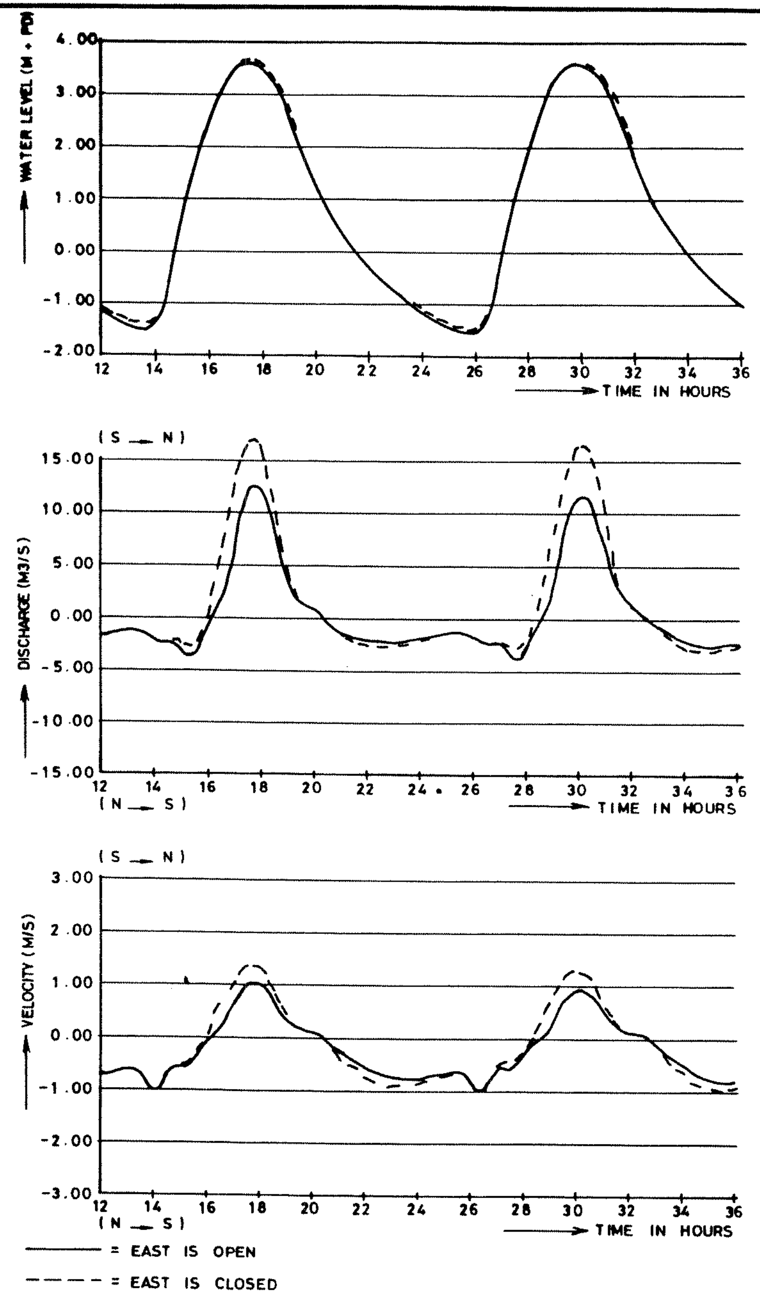
(a) An initial increase of 35% in the maximum current velocities.
(b) A substantial increase in head difference during high water-levels at both sides of the closure-gap. This implies higher velocities in the closure-gap, which will dictate the use of heavier closure elements.
(c) Increased head differences during low water-levels (though to a lesser extent than during high water), with similar implications.

The head differences in both cases (East open or closed) for a closed western channel have been given in Figure C.19.

C.4.3 Conclusion

Construction of the entire cross-dam could take place in a relatively short construction period, or in phases, with a considerable time interval between the closures of the two main channels. In the former case, a total construction time of approximately three years would be possible, with at the most one year between the two main closures. This time is very short compared to the time needed for accretion, reclamation and development of new land at both sides of the future cross-dam. Therefore it is strongly recommended that the (technically) most favourable closure sequence be selected, i.e. first West and then East.

If phased implementation is chosen the following technical aspects should be considered when deciding about the construction sequence of the closures:



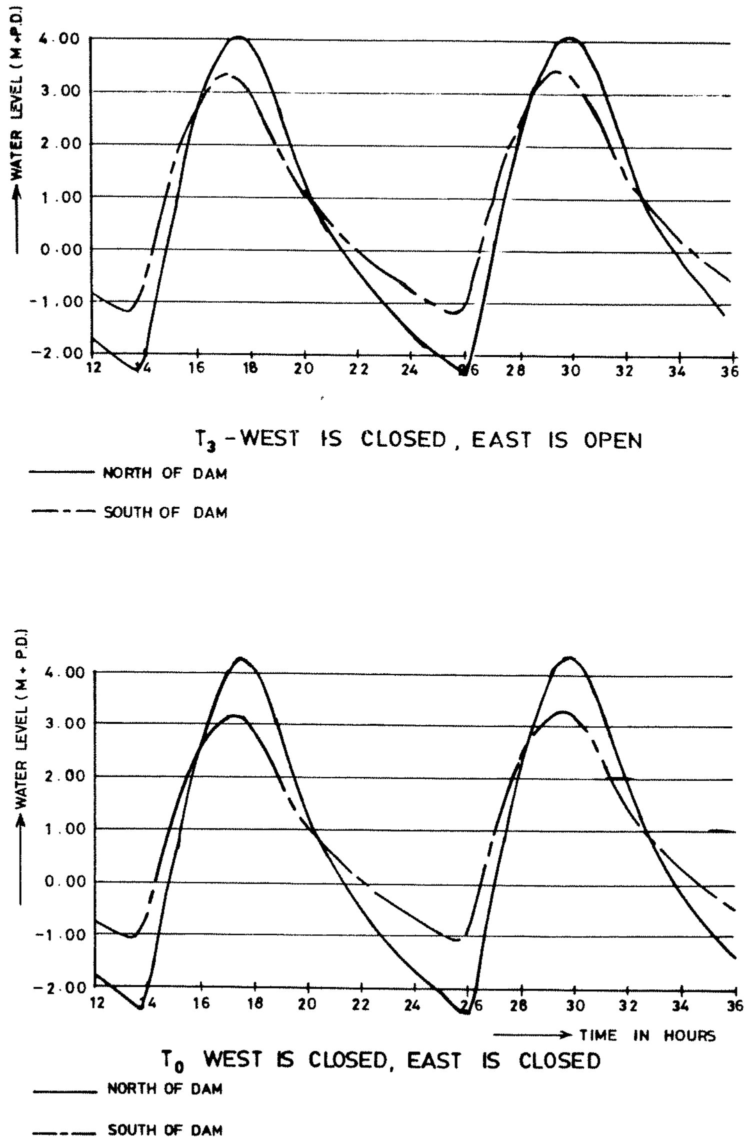
SEQUENCE OF CLOSURES-WATER LEVELS, DISCHARGES, VELOCITIES IN OPEN WESTERN CHANNEL

T_0/T_{12} - WEST

SPRING TIDE, 4.70m AT SATAL KHAL

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG.C18



SEQUENCE OF CLOSURES-WATER LEVELS AT BOTH SIDES OF WESTERN DAM AFTER CLOSURE (East open or closed)

T_3/T_6 - WEST

SPRING TIDE, 4.70m AT SATAL KHAL

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG.C19

(a) If the western channel is closed first:

- The eastern closure will not become more difficult.
- Erosion at the coast of Sandwip will hardly be influenced; it may proceed at the same (or slightly lower) rate.

(b) If eastern channel is closed first:

- The western closure will become much more difficult. In view of the substantial scour to be expected at the closure site the volumes of material needed for the closure dam will also increase substantially.
- The erosion along the western coast of Char Pir Baksh will increase significantly, as the increased discharges through the western channel will reinforce the present erosion pattern.

For the purpose of the feasibility study it has been assumed that the whole project will be realised as a continuous process and that during the construction of the cross-dam the western channel will be closed first.

C.5 Closure methods

The Sandwip cross-dam has an overall length of 22 kilometres. A large part of the dam (16.7 km) is located on the chars and on shoals which run dry during low tide. The actual closures concern the four tidal channels which cross the alignment of the dam. The purpose of the closures is solely to stop the tidal flow. The cross-section of the closure structure will form part of the final profile of the cross-dam (Chapter C.10).

In the following subsections possible closure methods will be reviewed. They can broadly be divided into:

- gradual closures;
- sudden (or instantaneous) closures;
- sand closures (as a special form of a gradual closure).

These methods are used in situations where a river or tidal channel has been narrowed to such an extent that the current velocities in the remaining channel do not exceed maximum permissible values. Detailed considerations on the initial narrowing operations are given in Chapter C.7. Prior to or during this narrowing the channel bed has to be protected in the axis of closure, for instance by placing mattresses. This is to prevent scouring of the channels, particularly in the gullies, due to higher current velocities (which in turn are a consequence of the narrowing operations on the shoals).

C.5.1 Gradual closures

A gradual closure aims at reducing the cross-sectional area of the tidal channel by placing current-resistant elements (such as rocks, boulders, concrete blocks or gunny bags). The tidal channel can be reduced in flow profile (Figure C.20) in three ways:

- horizontally from one or both banks
- vertically
- by a combination of horizontal and vertical methods.

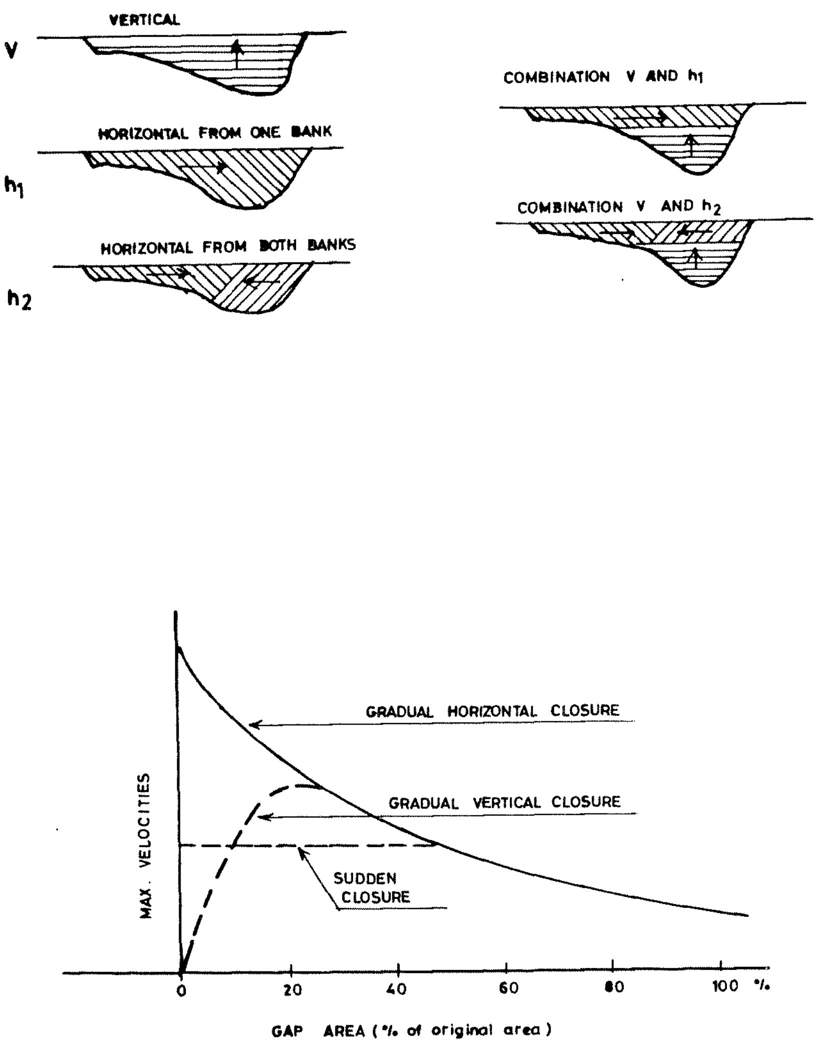
In Bangladesh there are examples of both horizontal and vertical closures. The selection of any of these techniques depends largely on:

- materials available for closure (natural or pre-fabricated, individual elements or assembled to larger units);
- hydraulic, soil-mechanics and meteorological conditions;
- scour and erosion hazards;
- characteristics of materials and structures placed in the closure-gap to fulfil a final function in the dam;
- geometry and dimensions of channels to be closed;
- labour available and (special) equipment to be applied;
- construction period and cost;
- risks involved;
- local experience.

The principal difference between a horizontal and a vertical closure follows from the maximum current velocity to be expected in the decreasing closure gap. For a horizontal closure the maximum velocity is proportional to the square root of the maximum head difference in the gap. This maximum velocity is reached just before closure. In the case of the tidal channels to be closed for the Sandwip cross-dam, the tidal movement takes place at both sides of the dam and head differences over the dam are due to phase differences of the tidal waves in the Sandwip and Hatia channels. For the western and eastern closures the maximum velocity would be around 5.5 m/s, assuming that the closure would be effectuated in the winter season.

For a vertical closure the maximum velocities are restricted by the occurrence of the critical flow situation over the sill. The maximum velocity in this situation is proportional to the square root of the upstream water depth relative to the sill crest level. After reaching the critical flow situation, raising the level of the sill means decreasing the water depth over the sill. This implies a decrease of the velocity, as can be seen in Figure C.20. For high tidal ranges (as occurring at the cross-dam site) this critical flow situation is reached at a lower sill level than for small tidal ranges.

The stability of materials placed in the closure-gap is, apart from weight, size and shape, determined by location and current velocity. The higher the current velocities, the smaller the resistance to the current of a block of a particular size. This demonstrates the need to keep current velocities in a closure-gap as low as possible.



COMBINATIONS OF HORIZONTAL AND VERTICAL CLOSURES - MAXIMUM VELOCITIES IN CLOSURE GAPS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.20

C.5.2 Sudden closures

The idea behind a sudden closure is to close a final remaining gap within a very short time. In this manner the high current velocities, as associated with gradual horizontal or (to a lesser extent) vertical closures, can be avoided. Sudden closure can take place either by instantaneous blockage of the whole area of the closure gap or by "building-up" a barrier against the tide in the time lapse between low and high water-levels. The instantaneous blockage can be done by sinking ships or caissons, or by closing the gates of a structure placed in the closure-gap. An example of building up a barrier against the tide is the Feni closure (Bangladesh, 1985).

C.5.3 Combinations of gradual and sudden closures

Apart from the combination of horizontal and vertical closures (both gradual closures) it is also possible, and in fact in many cases desirable, to combine gradual with sudden closures. In practice the tidal channel is first of all partly closed by a gradual horizontal and/or vertical closure, followed by a sudden closure. The gradual closure is made as far as current velocities permit, and to prepare a sill for the sudden closure to take place. The sudden closure is then realized for instance by placing (sluice) caissons or constructing a closure bund (Feni closure).

C.5.4 Sand closures

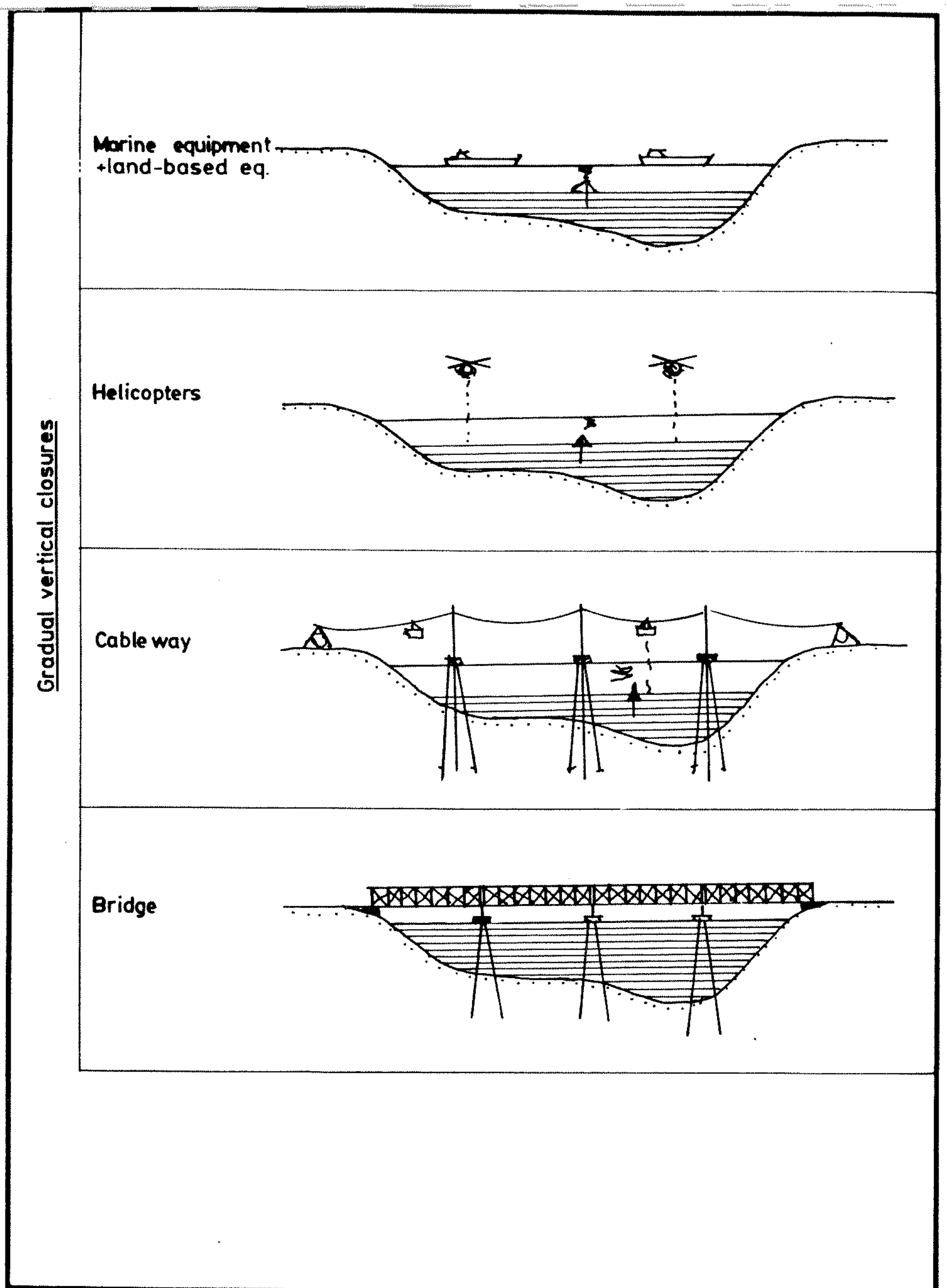
For the gradual and sudden closure methods discussed above it is essential that elements placed in the closure-gap are stable enough so that the currents cannot transport the elements. (Some damage may be acceptable for gradual closures under exceptional circumstances). This requirement of stability is less rigid for a sand closure, where sand is "deposited" in the closure gap by one or more high-capacity suction dredgers. For a sand closure the stability of a single element (sand particle) is not the criterium for success. The success (or failure !) is however governed by:

(a) the mean diameter (D_{50}) of the sand particles, and
(b) the capacity of the dredger(s) compared to the losses due to the currents at the head of the completed dam section.

For a sand closure to be successful it is necessary that the maximum current velocities in the closure-gap (which is decreasing in size) would not become too high: 2.0 - 2.5 m/s.

C.5.5 Figures

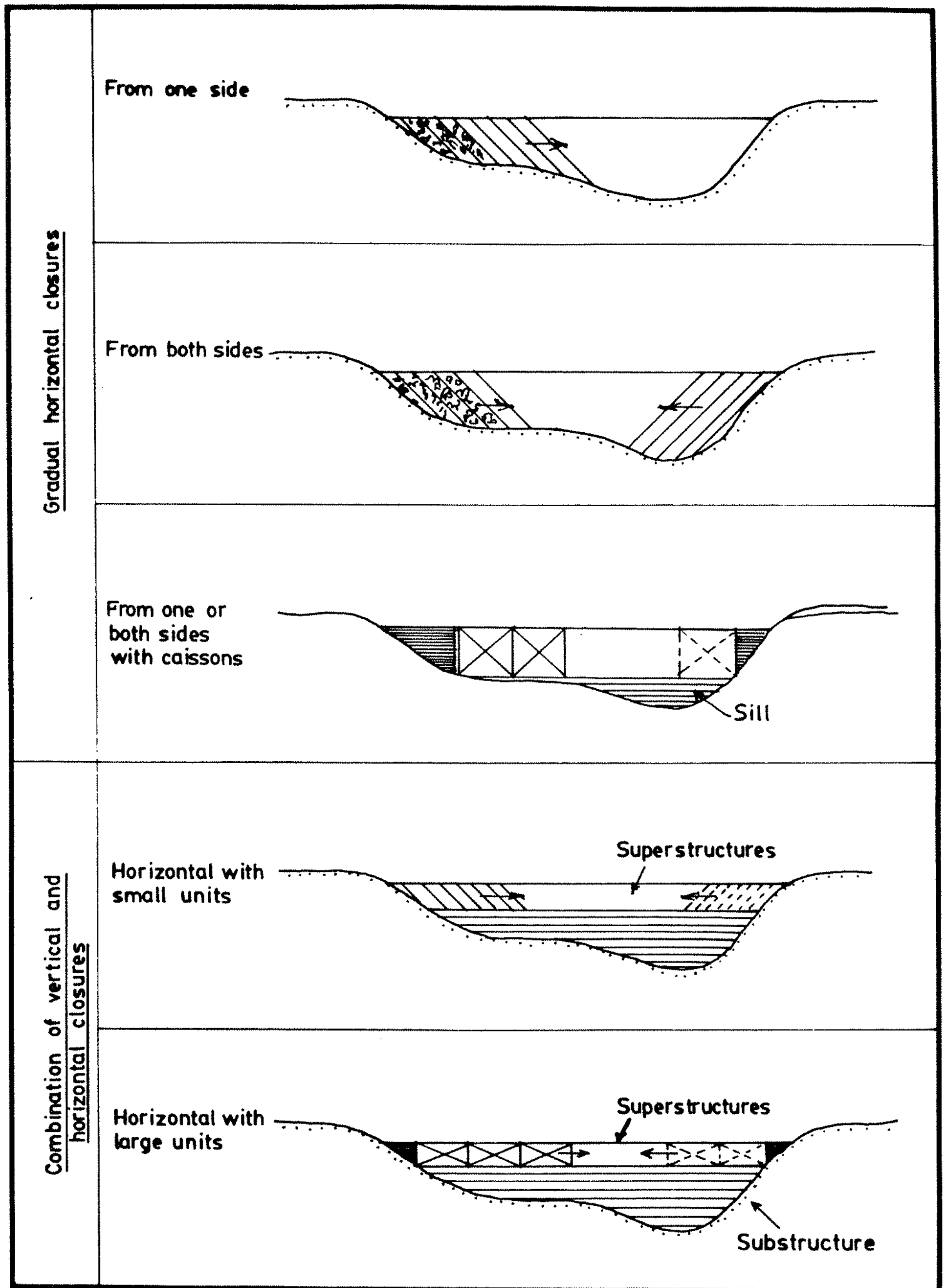
Figures C.21, C.22 and C.23 illustrate the various methods and combinations of methods discussed above.



GRADUAL VERTICAL CLOSURE METHODS

SANDWIP CROSS-DAM FEASIBILITY STUDY

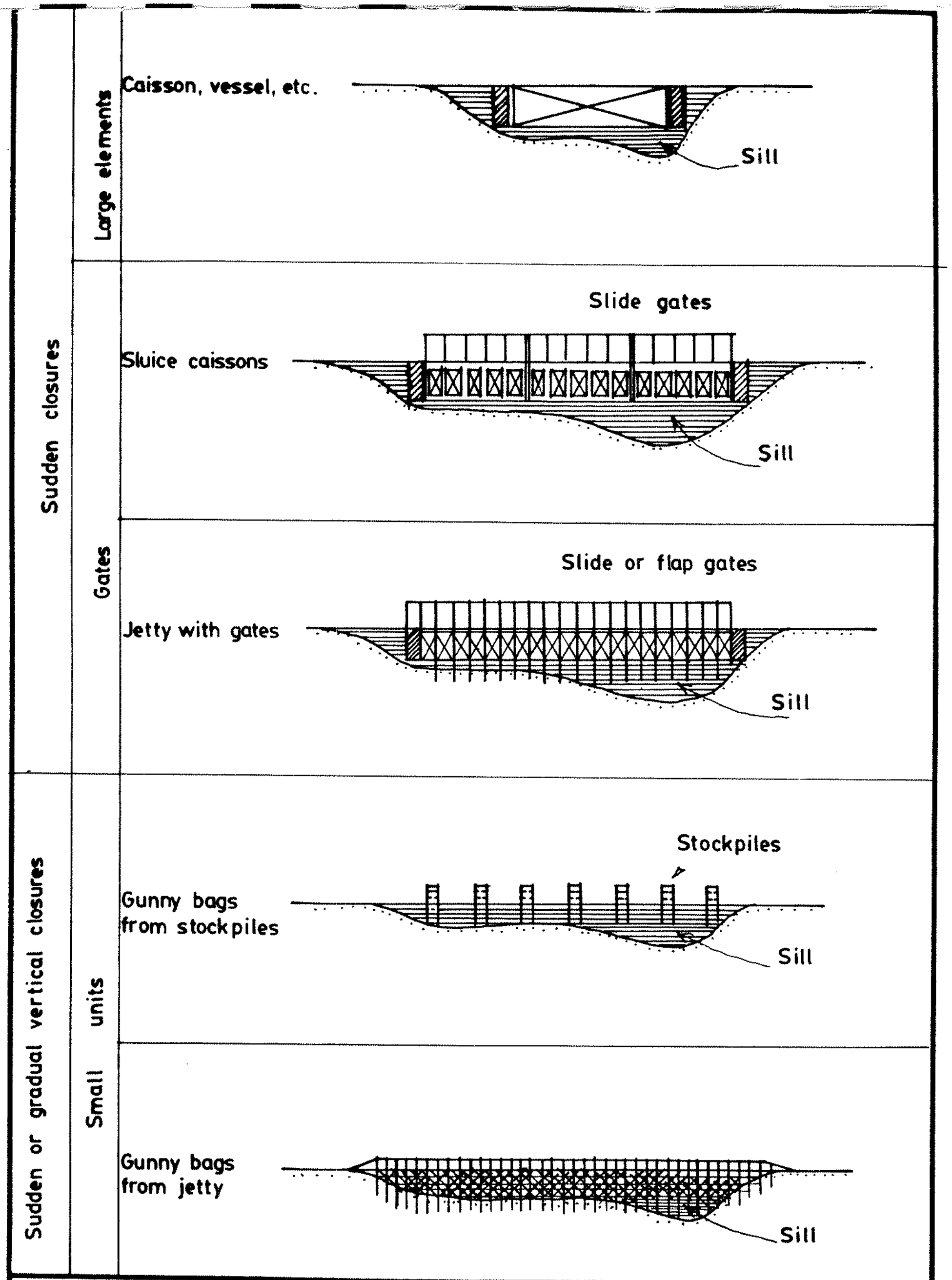
FIG. C.21



GRADUAL HORIZONTAL CLOSURE METHODS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.22



SUDDEN CLOSURE METHODS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.23

C.6 Construction materials

A first survey of the construction resources situation in Bangladesh was undertaken in December 1984. In the course of 1985 staff of BWDB conducted tests on the durability of bamboo and on the stability of earth-filled gunny bags. BWDB staff further undertook extensive surveys of the Chittagong area, the Noakhali area, Sandwip island and Char Pir Baksh, to report on potential borrow areas for clay and potential quarries for rock. Their findings are incorporated in the following sections.

Meanwhile preliminary designs of the closure-dams indicated that hard materials, such as rock, concrete or perhaps composite units (for instance brick-mortar blocks) would be required in large quantities for one or more of the alternative closure methods. If rock is available a total volume of about 600 000 m^3 may be required. Knowing that rock in Bangladesh is relatively scarce, it was anticipated that the supply of such a large volume of rock might be critical and for this reason much attention was directed towards the question from what sources in Bangladesh rock can be obtained.

C.6.1 Materials for bed protection

C.6.1.1 Bamboo

There are about 20-30 species of bamboo in Bangladesh, 12 of which occur only in village groves. There are two village species: Bambusa balcoa and Bambusa tulda. Bambusa vulgaris is also widespread, but it grows mainly in the Chittagong districts, while Bambusa arundinacea is less common and the least abundant. Those four species are the main bamboo species common in Bangladesh.

With the exception of B. tulda, which has smaller culms, all these species have clumps and culms up to 15-20 m tall. All four species have thick walls of 1.5-3.0 cm. The thick-walled muli bamboo is also found in the Chittagong Hill Tracts.

Bamboo will be applied for constructing the bed protection mattresses. About 1 300 000 m^2 bed protection mattresses will have to be laid, requiring some 700 000 bamboos to be supplied in the course of 2 years. It is considered that such a quantity can be supplied in Bangladesh bearing in mind that the annual crop in Bangladesh is 1 million tonnes (about 25 million bamboos).

Marine borer, if present in the waters near Sandwip, could rapidly destroy the bamboos for the bed protection mattresses. However, because of the seasonal fluctuations of the salt-content of the waters near Sandwip from almost fresh to almost pure sea-water, it is highly unlikely that marine borers are present in the area, as they cannot survive these fluctuations. In the spring of 1985 a test was conducted in which a bundle of bamboos was ballasted with stone and placed under water off the NW-point of Sandwip. After about one month the bamboo was retrieved, and no marine borers were found.

C.6.1.2 Reed

In mattresses made of "geo-textiles" and bamboo fascines, reed is used in rolls tied cross-wise onto the bamboos. They are used to add buoyancy to the mattress. Reed weighs about 130-180 kg/m^3, depending on diameter and moisture content: green reed is heavier than dried reed. For mattress construction dry reed is preferred.

For the Sandwip cross-dam the number of reed rolls required may be in the order of 800 000 (for 1 million m^2 mattress). Reed comes mostly from the south-west of the delta area. It is considered possible to supply the required quantity of reed in the estimated two years of construction time available.

C.6.1.3 Bricks

A great number of brick factories are found in the Chittagong area and on the Noakhali mainland. Hence ample quantities of brick can be obtained. As an alternative to boulders, brick can be used to ballast the bed protection mattresses at the locations not subjected to severe current attack (i.e. the shallow sections of the dam). Due to the lower specific gravity of brick (1.7 t/m^3) compared to rock (2.6 t/m^3), rock is to be preferred for ballasting when current velocities are high, such as in the closure-gaps. Nevertheless, if the procurement of rock and in particular the rate of supply of rock cannot meet the requirements, brick can be used for about 400 000 m^2 mattress (type A) requiring 150 kg/m^2 brick or about 42 bricks/m^2.

A heavier unit weight can be attained by making brick/mortar blocks of say 0.6 x 0.6 x 0.5 m. Such units, which are often applied by the BWDB, weigh more than 300 kg and require equipment for handling and placing. To save on mortar and labour costs it was suggested to manufacture bricks of a larger size than the standard size (of about 8 x 11.4 x 23.4 cm). Technically this is possible. Drying time of the "green" stones would have to be extended. Firing time at 800-900 °C need not be extended. Generally, however, larger bricks are heavier and therefore more difficult to handle manually.

C.6.1.4 Geotextiles

It will be necessary to apply some 1 million m^2 geotextiles for the construction of the bed-protection mattresses. These geotextiles must fulfil certain specifications concerning mesh and strength. This type of geotextile is not produced in Bangladesh and will have to be imported.

C.6.2 Materials for dam construction

C.6.2.1 Gunny bags

With Bangladesh being the world's largest producer of jute it is hardly surprising that the application of jute bags (also called gunny bags) in Bangladesh is well-tried in numerous small closure-dams.

The bags are filled with clay. A standard size bag filled with clay weighs about 45 kg. For the Sandwip cross-dam it is estimated that more than 5 million bags will be used. Partly, gunny bags will be applied in the earthfill dam sections built over the more shallow areas. Clay-filled gunny bags will also be used in two minor gully-crossings east of Char Pir Baksh. Bangladesh produces about 1200 million bags per year (figure derived from 1983 statistics), so there should be no difficulty in supplying gunny bags for the cross-dam.

Although clay-filled gunny bags have been used on numerous occasions in Bangladesh, they have never been tried for a long period under the circumstances expected at the Noakhali/Sandwip cross-dam where the current velocities in the main closure gaps will reach 3-4 m/s. Experience has shown that under severe current conditions clay-filled gunny bags will wash out, but no accurate information on the behaviour in strong currents was available. In view of the design for the Sandwip cross-dam it was realized that tests were necessary to ascertain the current stability of clay-filled gunny bags, and in the course of 1985 tests were set up and conducted at Char Batir Tek in Bangladesh, and Lith in the Netherlands.

The tests at Char Batir Tek, Noakhali District, were carried out in April 1985. High current velocities up to about 3 m/s could be generated in the Water Control Structure of BWDB located there. In a first test, gunny bags were dropped in the current just downstream of the gates, which resulted in practically instantaneous removal of the bags. On a second occasion bags were placed downstream at slack water, but after building up of the current velocity to about 2.6 m/s the bags began to be washed away.

The tests at Lith were conducted in a measurement-flume located alongside a sluice in the river Maas. Test conditions could be controlled accurately by regulating the height of the sliding gates. In this flume current velocities of up to 5 m/could be generated. The tests conducted at Lith confirmed the earlier findings at Char Batir Tek.

The so-called "consolidation" effect, ascribed to the cohesiveness and plasticity of clay, indeed serves as a minor positive factor in the interlocking effect of the bags. Bags move at a later stage than is to be expected from application of the usual formulae, which do not take this consolidation effect into account. However, a deciding factor in the stability of the clay-filled bag is reached when turbulent conditions start sucking out the clay through the pores of the bags and great brown clouds appear in the water. As the bags become lighter in the process they will then be moved by the current. For all practical purposes a current velocity of 2.6 m/s should be seen to be the maximum current velocity in which clay-filled gunny bags are to be applied. Greater stability is obtained by placing bags on a mattress with willow fascines forming box-like compartments for the bags. Nevertheless, when the current reaches 3 m/s the earlier described wash-out of clay becomes the decisive factor.

Experience with the durability of gunny bags was obtained during the construction of the Feni dam. It was observed that clay-filled gunny bags in dry stockpile were susceptible to a form of bacteriological rot or fungus growth. The jute lost its strength and would tear easily. The durability and rot resistance of the jute can be improved by pressure-impregnating it with a chemical such as copper-chromium-fluor. To check this, bags impregnated with this chemical were taken from the Netherlands to Bangladesh in January 1986, where they are currently being tested in the LRP Pilot Polder Char Baggar Dona. On 11 February the treated bags were placed in a stockpile of 6x6x6 bags otherwise untreated. The degree of weathering and/or rot of the untreated bags will be compared with the treated bags. Samples will be tested in a laboratory in Europe.

C.6.2.2 Clay

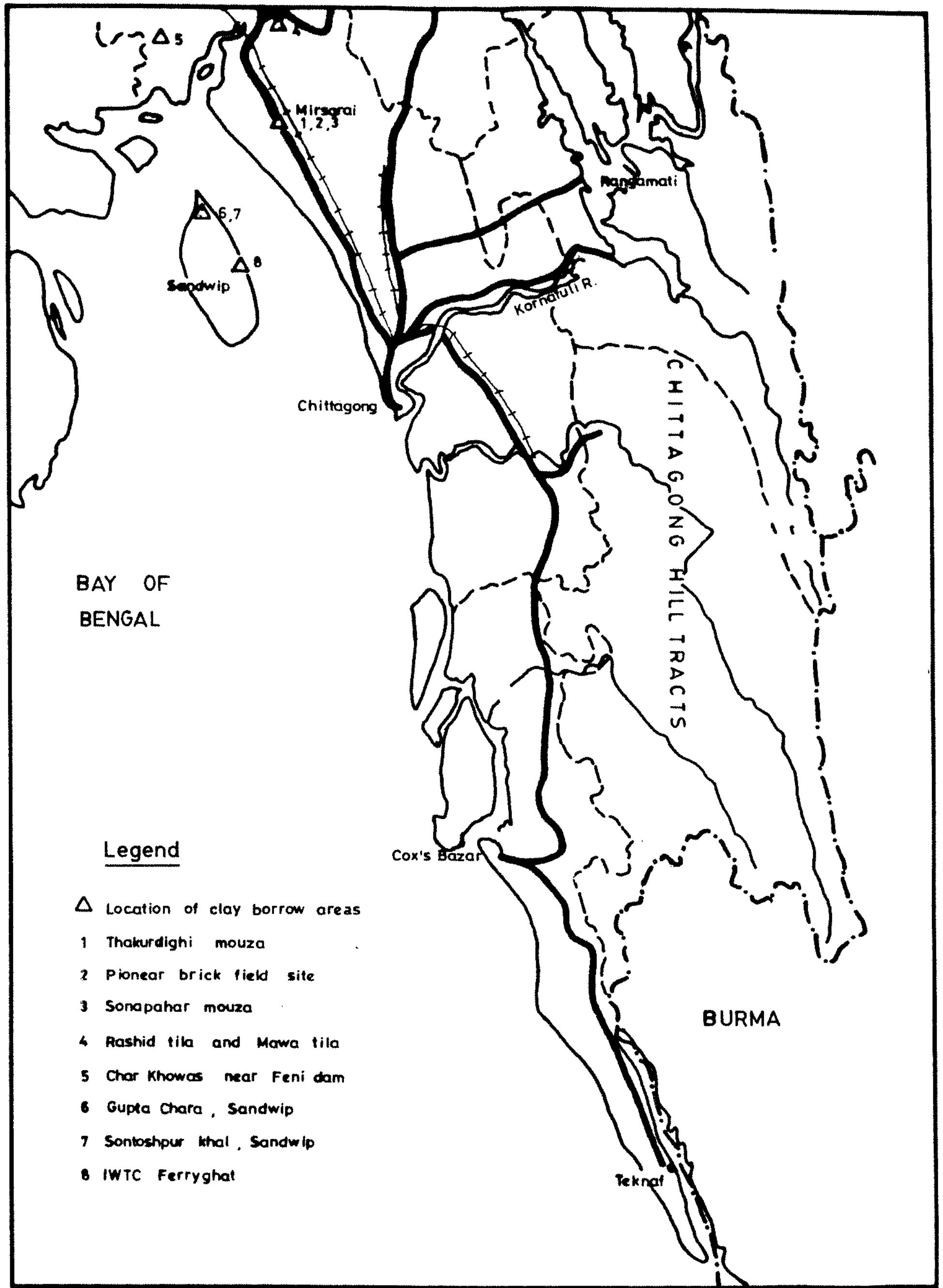
For the construction of the cross-dam some 250 000 m^3 of clay will be required: as fill in gunny bags, and for the slope protection. In the course of 1985 BWDB-staff conducted an extensive survey of the Chittagong area, Char Pir Baksh and Sandwip island. Fig. C.24 shows the locations of potential clay borrow areas visited. The conclusions can be summarized as follows:

- No suitable clay (with a clay fraction component of more than 15%) has been found at Char Lakhi or at Char Pir Baksh.

- Clay is found at Sandwip at its north-eastern tip in the area of Santoshpura Khal (just inside the embankment) and at Gupta Chara Khal near the ferryghat (outside the BWDB embankment).

- Clay can be borrowed from several hillocks situated east of the main Chittagong-Feni trunk road and also at some locations near the Feni dam site on the right bank of the Muhuri river. In particular the hillocks (tilas) are quite suitable for borrowing clay; the farmers would welcome the removal of these hillocks, as the remaining land will become level and more suitable for cultivation. Transport costs are higher, however. These hillocks may vary in size from about 0.4 to 4 ha, and may be 10 to 30 m high containing some 20 000 m^3 to 400 000 m^3 of clay.

Samples of various clay borrow areas were analysed. Fig. C.25 and C.26 show typical grain size distribution curves for the clay found near the Feni dam (Char Khowaz) and at Sandwip (Gupta Chara Char near IWTC ferryghat).



POTENTIAL CLAY BORROW AREAS

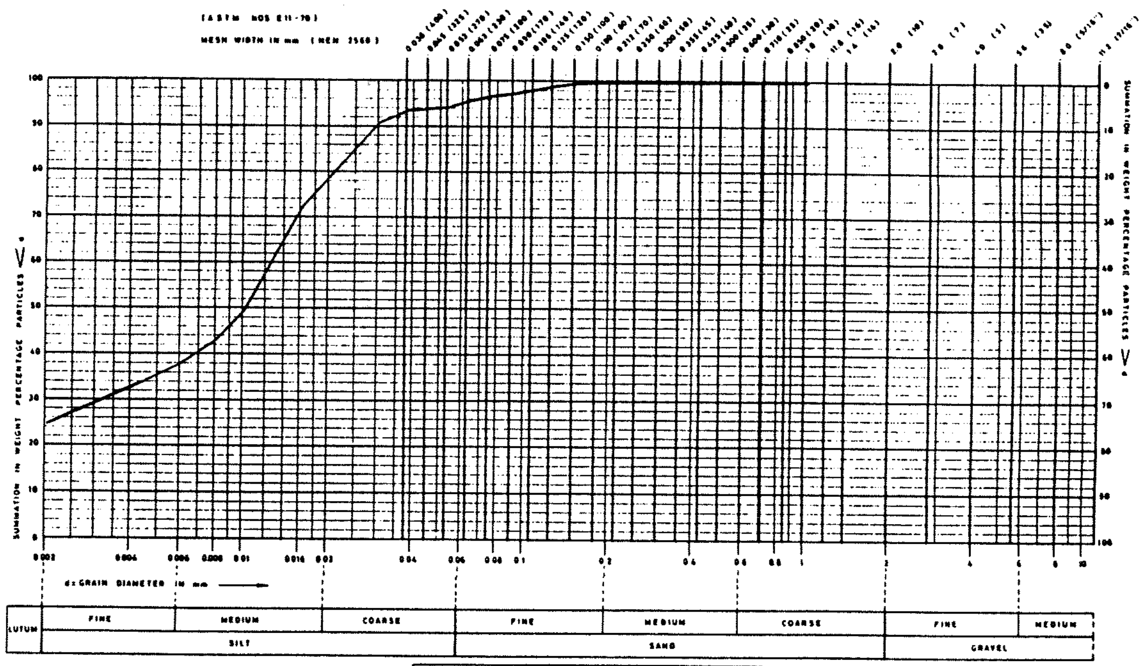
SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG.C24

SANDWIP CROSS-DAM FEASIBILITY STUDY

GRAIN SIZE DISTRIBUTION OF CLAY
AT CHAR KHOWAZ

FIG. C.25



no Atterberg limits due to
small sample.

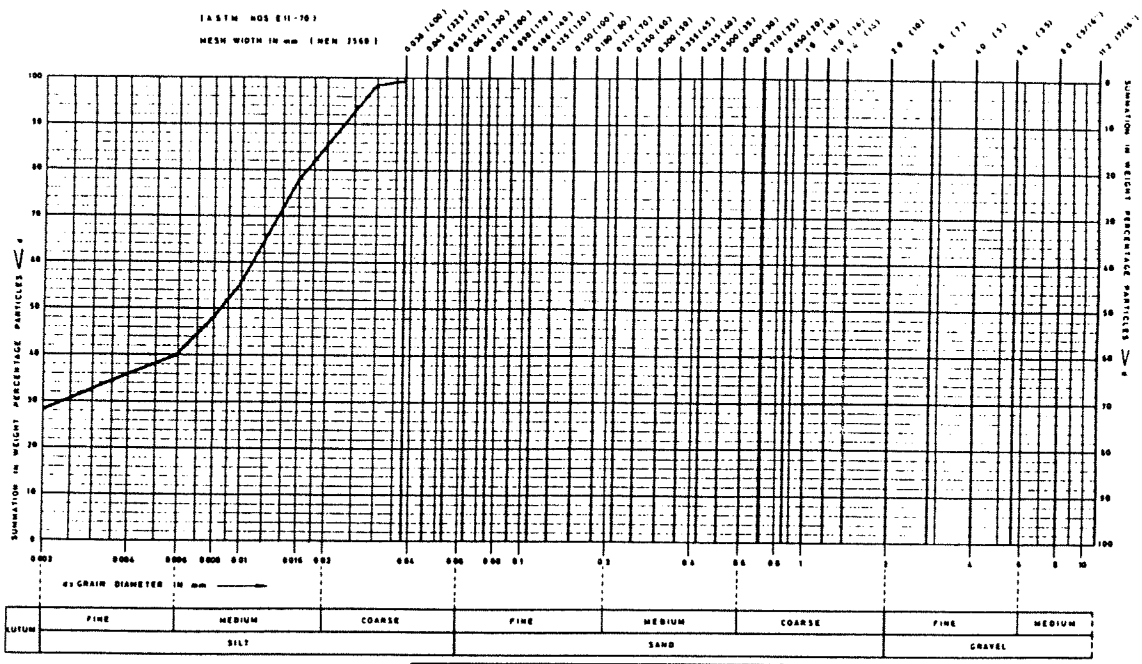
SPEC. GRAVITY: 2,734t/m3

	BORING	SAMPLE	DEPTH IN m REFERRED TO
——		2	

SANDWIP CROSS-DAM FEASIBILITY STUDY

GRAIN SIZE DISTRUBUTION OF CLAY
AT GUPTACHARA

FIG. C.26



no Atterberg limits due to
small sample.

SPEC. GRAVITY: 2,734t/m3

	BORING	SAMPLE	DEPTH IN m REFERRED TO
——		1	2'

C.6.2.3 Earth fill

The approximate earth fill quantities needed for the different sections of the cross-dam are given below:

- western section, dam on the shoals	480 000 m^3
- eastern section, dam on the shoals	590 000 "
- central section, dam on the shoals and chars	852 000 "
- central section, central closures	359 000 "
Total	2 281 000 "

The procurement of the quantities will pose no problems, as suitable earth is abundantly available.

For the dam on the shoals in the western section earth can be borrowed from the Char Lakhi side, about 240 000 m^3 from charland (= khas land) and about 240 000 m^3 from embanked land. Khas land is government property, but for the borrow pits on embanked land about 8 ha will have to be acquired. The acquisition costs are included in the rates.

For the dam on the shoals and chars in the central and eastern sections, and for the central closures, earth can be borrowed on Char Pir Baksh and on the two chars east of Char Pir Baksh, both along the alignment of the cross-dam and from borrow pits. At present all land is charland and thus khas land; no leases should be given along the dam alignment and near the east bank. The total area for this dam section needed for borrow pits is approximately 30 ha, of which 8 ha on Char Pir Baksh and the remainder on the two chars east of Char Pir Baksh.

The excavation of earth need not negatively affect the area. Positive use can be made of these borrow areas provided they are laid out and designed to form fish ponds.

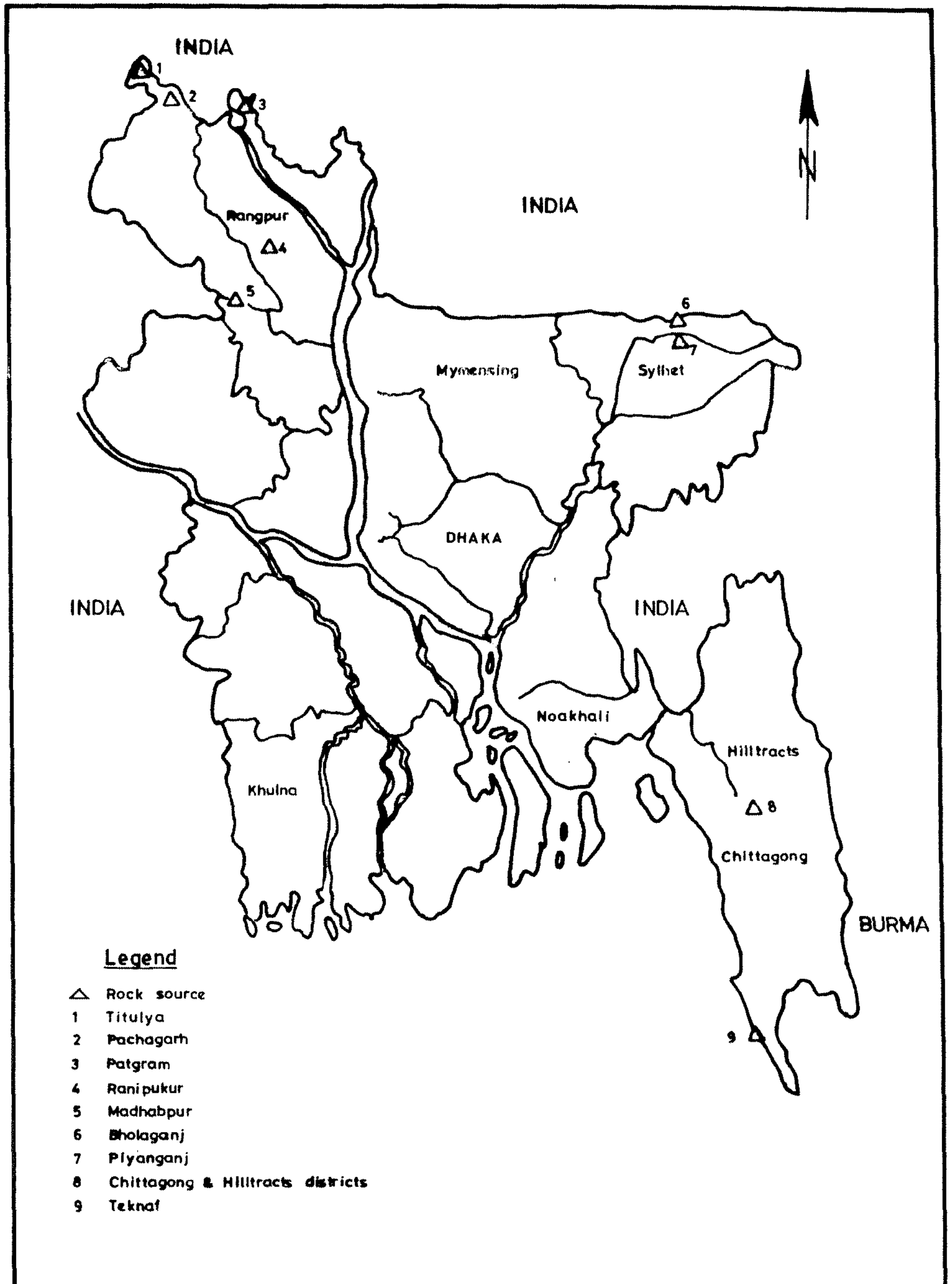
C.6.2.4 Rock

Some 50 000 m^3 of boulders will be needed for ballasting the mattresses, while about 500 000 m^3 of rock will be required for constructing the sill sections of the dam and for ballasting the mattresses in the most heavily attacked areas. Hard rock is found in the following three regions (Fig. C.27):

- In the upper north-west of the country: near the Indian border at Titulya, Pachagarh and Patgram, and more inland at Ranipukur near Rangpur and at Madhabpur.

- In the Sylhet area at Bholaganj and Piyanganj near the Indian border.

- In the Chittagong Hill Tracts.



ROCK SOURCES IN BANGLADESH

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.27

As regards the rock found in the north-west of Bangladesh, it must be concluded that this rock is unsuitable for the Sandwip cross-dam. From information obtained from the Mineral Corporation (Petrobangla) this area produces small gravel-stone only. This is used locally in the region and not elsewhere in Bangladesh. Larger rock is required for ballasting the mattresses or for building up the sills.

The rock from Sylhet, which comes in the form of boulders collected from the river beds, will be suitable for ballasting the mattresses. The Sylhet boulders may be found in sizes up to about 0.3 m diameter. Each rainy season new boulders are swept down the rivers into the Sylhet area from India across the border. The available volume of these boulders is almost inexhaustible. The Sylhet boulders are not suitable for sill construction because they are too small, and their round shape offers less resistance to the currents than broken rock.

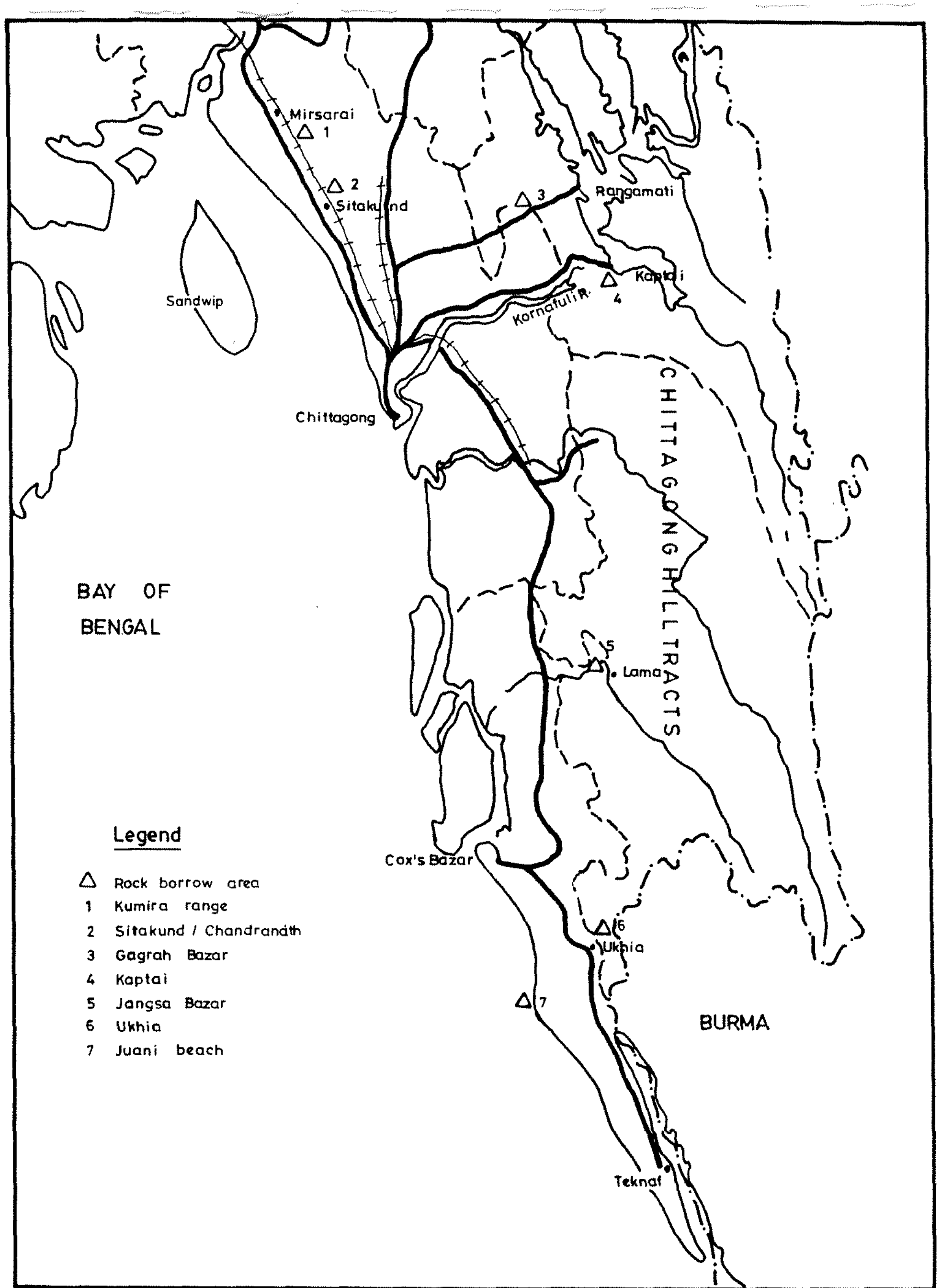
An extensive survey was conducted in the Chittagong Hill Tracts, because their relative proximity to the cross-dam site would make shipping of rock from this area feasible. The results of the survey are described below.

In the area a large number of small-scale quarries are being operated extracting rocks (mostly sandstone or conglomerate) from the hill streams (called "charas") and in a few instances from hill sites containing large conglomerate rock formations embedded in the soil. At one location - Juani Beach, about 28 km south of Cox's Bazar - rock formations are found on the beach which disappear at high water but are exposed at low water. The borrow sites visited during the survey are shown in Fig. C.28 (Kumira range, Sitakund, Gagrah Bazar, Kaptai, Jangsa Bazar, Ukhia and Juani Beach).

The general impression obtained from the survey is that the sandstone and conglomerate from the Chittagong mainland are not attractive for use in concrete. Larger rock up to about 300 kg can be obtained from the river beds but the overall output and rate of production will not be sufficient to fulfil the requirements of the Sandwip cross-dam. Each source (i.e. each river bed borrow area) has a limited quantity of rock, while the remoteness of the sites usually requires construction of an access road with culverts and bridges, making the cost of rock extraction relatively high.

The area in general, and in particular the area bordering Burma, has as yet been poorly surveyed, and it is not impossible that massive rock deposits are present, which would offer the possibility of developing an extensive quarry. It would be useful to carry out seismic refraction surveys to assess this more definitely. Meanwhile, alternative solutions have to be considered as well, namely:

- making pre-cast concrete blocks; the cost of making these blocks lies in the same range as the cost of obtaining rock;
- importing rock, for instance from Madras (India), Thailand or Malaysia, which has a desirable high specific gravity; preliminary information indicates that rock can be imported at competitive prices.



ROCK SOURCES IN CHITTAGONG AREA

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.28

C.6.2.5 Concrete

Structural reinforced concrete would be necessary for the closures if caissons are selected for the final closure. Unreinforced concrete will further be used for making concrete elements for the slope protection of the dam. Furthermore concrete blocks may be made as an alternative to the application of large rock in the sills of the dam. closure sections.

The best quality sand for structural concrete is river bed sand from Sylhet. As it is too coarse it is usually mixed with finer sand. Lesser quality sand which is finer and may contain impurities (such as silt) is dredged from the rivers at many sites elsewhere. Medium-coarse sand can also be obtained from the outer bay, the inner bay and the Gupta bay in the Karnafuli river in Chittagong, but at some places in the Karnafuli river the silt content may be too high.

With regard to coarse aggregate, the best quality concrete is achieved by using Sylhet chippings. For lower grade concrete Sylhet shingles or other river gravels can be used. It was found that the rock (sandstone or conglomerate rock) from the Chittagong Hill Tracts is often unsuitable (too brittle) for concrete. As is the case with sand, Sylhet and other river gravel is abundantly available and enough of these aggregates can be supplied. Brick chippings are often used as aggregate for concrete, but the low density of bricks is a disadvantage when they are used in concrete blocks for the sills.

Cement produced from limestone is made in factories in Chittagong and in Chatak (Sylhet District). (In Chittagang cement-clinker is imported and ground to cement ready for use). The production over the past years has been an average 300 000 tonnes per year. Cement is also imported from Japan, Thailand and Indonesia. It is to be expected that part of the cement necessary for the concrete for the Sandwip cross-dam will have to be imported.

Reinforcing steel will be needed if caisson closures are chosen. Concrete blocks for the sills and concrete elements for the slope protection will be of non-reinforced concrete. There is a steel works in Chittagong producing mild steel rods. Its annual production has declined to 6835 t (1983-1984 figure).

C.6.2.6 Gabions

An attractive alternative to the use of heavy rocks in the core of the dam is the use of gabions, i.e. wire-mesh cages filled with bricks. The cubical blocks so formed are dumped from barges. The steel for the wire mesh need not be galvanised or plastic coated because the brick gabions will be covered in subsequent stages of the work. The curve may be about 4 mm in diameter.

C.6.3 Timber for structures

The construction of the cross-dam will require very little timber. Supplying this small quantity will present no problems in Bangladesh, which produced on average 450 000 m^3 of timber during the recent years. The main forest areas are in the districts Khulna (Sundarbans), Sylhet (Pathania) and Mymensingh (Madhupur Jungle) and in the Chittagong Hill Tracts.

'Bullah' piles can be of Sundari, Gorjon or Jarul wood. They come mostly in lengths of 6 m (20 ft) but piles can be 11 m (35 ft) long with diameters of up to 23 cm (9 inch) at 1/3 of the length from the top. The R.E.B. (Rural Electrification Board) uses these Bullah piles as poles for overhead power and telephone lines.

C.7 Pre-selection of closure methods

C.7.1 General approach

Analysis of various computer runs as presented in Section C.2 shows that the application of entirely horizontal closures for three out of four channels would result in high velocities in the last parts of the tidal channels to be closed. (Maximum velocities to be expected are: of the order of 5.5 m/s, assuming that the closure would be made in the winter season). Alternatively, the application of purely vertical closure methods would create an almost unmanageable situation during construction in view of the substantial lengths of the dam sections to be constructed. (partial) method, while avoiding the disadvantages, a combination of horizontal and vertical closure methods has been investigated in depth.

The final closure-gaps in the western and eastern channels could in principle be located either on the shoals or in the gullies to be crossed. To determine which method is to be preferred various computer runs were made, from which the following conclusion were drawn:

- initial narrowing on the shoals, prior to any other operation, which would reduce the cross-sectional area of the channel does not (or hardly) lead to more difficult flow situations during the subsequent closure operations in the channel(s), whereas
- initial raising of the channel bed, by constructing a sill, very soon leads to more difficult flow situations over the entire width of the channel.

It was, therefore, concluded that the final closure of the western and eastern channels would have to be undertaken in the gullies after first blocking the flow over the shoals.

The most attractive combination of horizontal and vertical closure methods requires the following main components:

(a) bed-protection over the full widths of shoals and gullies;
(b) low dam (or other structure) on the shoals;
(c) construction of sills in the two main gullies; the widths and crest levels of these sills depend on the closure method finally selected and the structural design thereof;

(d) - closure of these two main gullies to be made on top of the sill; the various options considered include gradual as well as sudden closure methods.
  - closure of the two central gullies to be made on top of the bed protection.

There is a strong inter-relationship between these main components as far as the individual lengths of closures and structural designs are concerned. Dimensions given must not be regarded as absolute figures, but are, among other things, related to the time (season) in which a particular component can be (or has to be) carried out. For instance, if the sill has to be constructed in the post-monsoon period (say October or November with larger tidal ranges) its length (= width of the closure-gap) may have to be increased when compared with execution in the winter season (say January or February with smaller tidal ranges). Alternatively, the sill elements would have to be heavier if early (= post-monsoon) construction is required for a particular closure method.

C.7.2 Pre-selection of closure methods

The pre-selection involves the selection of closure methods which, in principle, could be applied for one or more of the tidal channels. Each closure method results finally in a structure across the tidal channel to be closed. A distinction is made between substructure (all works up to the low water-level) and superstructure.

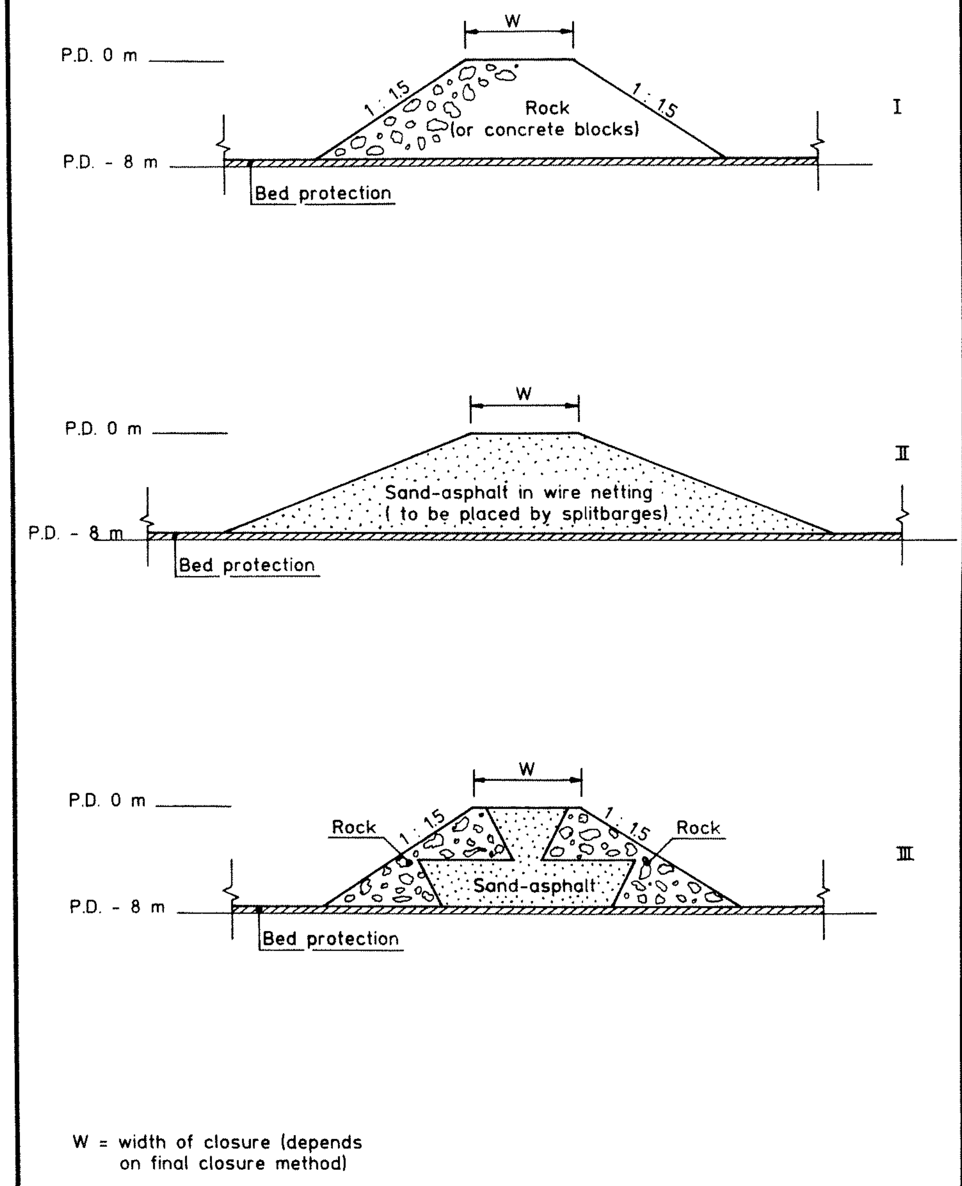
C.7.2.1 Substructure for various closure methods

A substructure for any closure always implies the application of a (partial) gradual closure method. A dam or sill is raised in horizontal layers on top of the bed-protection. The material can be placed in situ either by water-borne equipment, by a cableway or helicopter, or from a (temporary) bridge. It can be stated here already that water-borne equipment is to be preferred: a cableway or similar system is expensive, its design and manufacturing is time-consuming, its erection is cumbersome, it can only be used once, and it fixes the axis of closure at a very early stage, which goes at the cost of flexibility.

The dam or sill (a sill has a flat top, a dam does not have to) may consist of:

- rock, concrete blocks, or a combination thereof,
- boulders or bricks in gabions,
- sand-asphalt and rock (combination),
- gunny bags covered by mattresses.

The sketches in Figure C.29 illustrate some possible sill configurations.



POSSIBLE SILL CONFIGURATIONS
(SITUATION EASTERN GULLY)

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.29

C.7.2.2 Superstructure for various closure methods

A distinction has to be made between gradual vertical closures, gradual horizontal closures, and sudden closures.

(a) Gradual vertical closures

These closures can be effected either by placing successive layers of hard material (a-1) or by building up a cofferdam (a-2).

a-1 Structure made in horizontal layers by using land-based equipment.

'Hard' materials to be considered are:

- rock, concrete blocks or a combination thereof,
- boulders or bricks in gabions,
- bricks or boulders dumped into pre-fabricated concrete frames,
- sand-asphalt and rock (in combination).

a-2 Structure (called cofferdam) made by constructing a jetty on top of the under water substructure (the sill).

The jetty is transformed into a "cage" (by means of bamboos) from which dumped clay-filled gunny bags cannot escape. The cofferdam is gradually built up by dumping clay-filled gunny bags during 2-3 days. Transport and dumping of bags is carried out by manual labour. The method was successfully developed and applied in Bangladesh for closures up to a width of 210 m. The method will be considered for closing the eastern of the two central gullies near Char Pir Baksh having a width (in the deeper part) of 400 m. It is at present considered that no sill will be required and the closure would start from the bed protection upwards.

(b) Gradual horizontal closures

These closures can be effected either by building forward a (say up to 5 m high) dam having a steep front (b-1 and b-2) or by placing concrete caissons which are transported in floating condition and subsequently sunk on top of the sill (b-3).

b-1 Closure dam constructed from hard materials by using land-based equipment.

"Hard" materials to be considered are:
- rock, concrete blocks or a combination thereof,
- boulders or bricks in gabions,
- bricks or boulders dumped into pre-fabricated concrete frames,
- sand-asphalt and rock (in combination).

This method has been considered for closure of the two central gullies, but had to be rejected because of development of unacceptably high currents during narrowing operations.

b-2 Closure dam constructed by dumping gunny bags using manual labour

The dimensions of the western of the central gullies near Char Pir Baksh are such (width 600 m, max. depth PD+0 m) that a horizontal closure can be considered. A closure dam will be built forward from both banks by dumping gunny bags filled with clay directly onto the bed-protection. It should be borne in mind that this closure method is only feasible when the axis of closure is located more or less at the meeting point (if there is one) of the tidal waves entering this channel from Sandwip and Hatia channels. Obviously, in that case current velocities are low.

b-3 Closure by placing concrete caissons

The concrete caissons are pre-fabricated "boxes". Possible dimensions of such a box or element would be 9 m wide by 5.5 m high by 30 m long. The floating caissons are transported by tug boats and positioned prior to high water slack over the underwater sill on the closure axis. At slack water the caissons are sunk on the sill and ballasted with boulders or earth. The method will be considered for closure of the eastern channel. It was also considered for the western channel but here slack water already occurs at a water-level which is 0.7 m below high water, which renders placing of caissons technically unfeasible.

(c) Sudden closures

Sudden closures can be effected by taking a number of measures prior to the moment (or day) of sudden closure. It could be stated that the "super-structure" is pre-fabricated and becomes effective at a carefully selected closure day. The "structure" can be either a "sluice caisson" placed and sunk on the sill (c-1), or a kind of movable weir with flapgates (c-2) or a number of stockpiles which are "turned 90 degrees" by manual labour (c-3). In all cases the substructure (the sill) has to be prepared carefully in advance.

c-1 Sluice caissons

The sluice caissons are pre-fabricated concrete sluices which can float and which are provided with sliding gates. After placing one or more of the sluice caissons on a sill having a relatively low crest level the water is allowed to flow through the caissons rather freely until the moment of closure, when all gates of the caissons are closed simultaneously. The application of sluice caissons for the Sandwip cross-dam has not been considered attractive because:

- they are relatively expensive;
- the caissons have to be placed on a fairly horizontal sill, and it is difficult to make such a sill underwater without sophisticated water-borne equipment;
- as the caissons can only be placed on a rather low sill the shallower parts of the tidal channel have to be closed by another method in advance, which in the case of the Sandwip closures increases current velocities over the low sill to unacceptable levels.

c-2 Movable weir with flapgates

A comparable method is the use of gates fixed to a structural steel jetty. For this purpose piles will have to be driven into the previously constructed sill. The height of the sill depends on the type of jetty to be used. At the pre-feasibility level a jetty type structure with closure gates was suggested for the eastern closure.

A further analysis of the hydraulics of the channel system and workability has led to the conclusions that raker piles to support the gate structures should be avoided and that a sill level of PD + 0 m should be reached for construction of the supporting structure for the gates. A sketch design for a possible gate structure is given in Figure C.30.

The steel support structure will have to be constructed with land-based equipment in order to reach a sufficiently high production rate. It is estimated that the construction of the support structure and the installation of steel gates will take at least 9 months after completion of the sill. Upon completion of the gate structure, closure should take place instantaneously. In the case of 3 m wide gates, around 1000 gates will have to be lowered and fixed instantaneously!

After closure an earthfill dam has to be made, surrounding the gate structure. In view of the substantial construction time required for the earth dam, the gate structure should be strong and stable enough to withstand rather extreme monsoon and cyclone conditions. A collapse of one bay or gate may lead to progressive collapse of the neighbouring bays/gates.

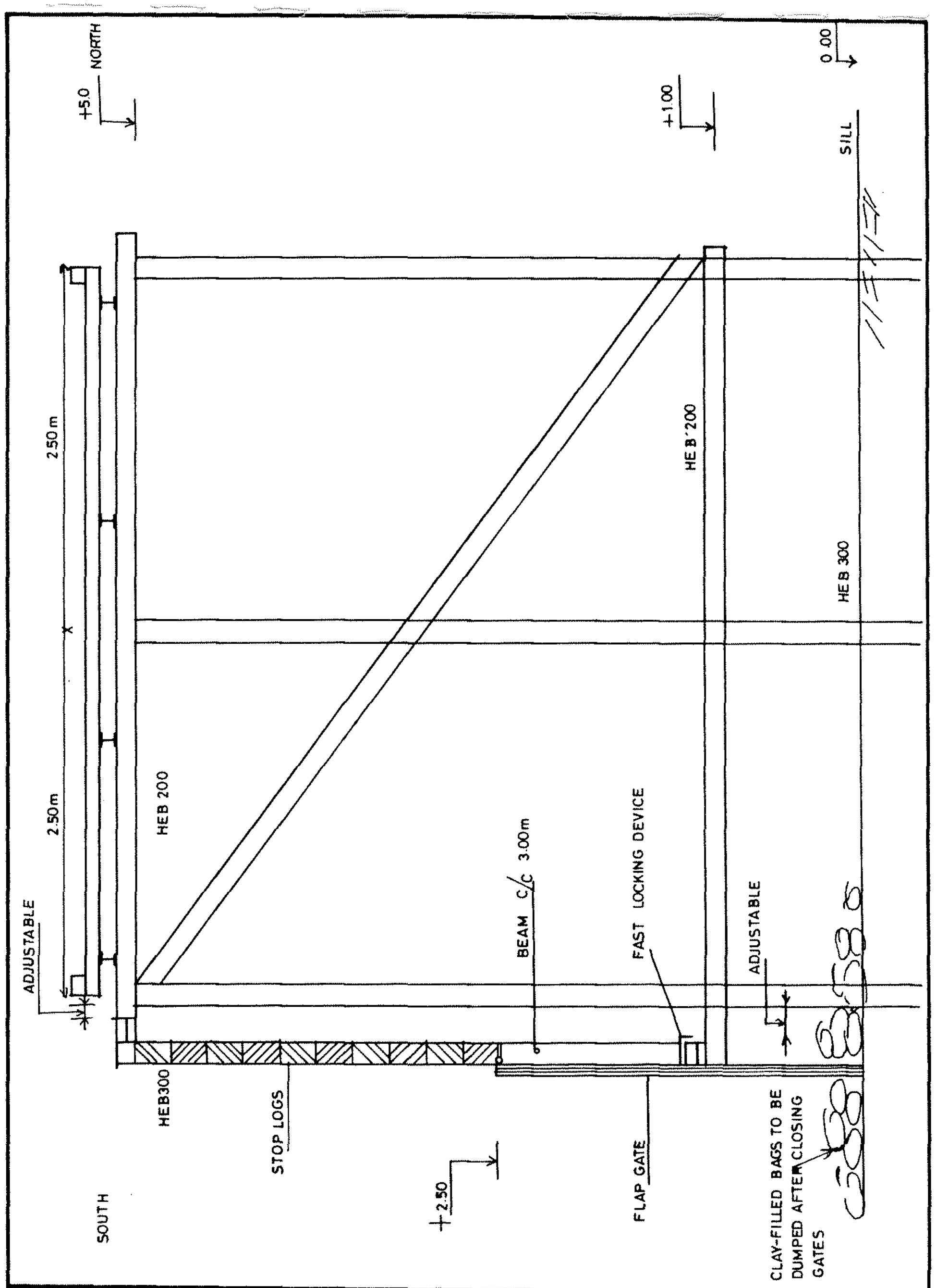
Compared with, for instance, a gradual stone closure described in point a-1 it has to be concluded that:

- a long execution time is required for the gates structure;
- after closure the gate structure will be exposed to the elements for rather a long time;
- an earthdam with slope protection is required (approx. 600 000 m^3 of earth, most of it to be excavated from Sandwip);
- risks during construction are rather high.

For all these reasons it was decided not to consider this closure method any further.

c-3 Closure bund made up of gunny bags

A special type of sudden closure was applied in 1985 for the Feni River closure dam. This closure was achieved with gunny bags which were placed over the entire river width (1300 m) by 12 000 labourers within a time span of 6 hours, when the river bed became dry during a neap tide. For a number of reasons it was not advisable to narrow the Feni River before the actual closure. The whole river width formed in this case the closure gap. Before the closure a sill had been made in the gullies to match the level of the shoals. In order to reduce the transport distance for the gunny bags during the closure operation, stockpiles had been made in the river at regular intervals (100 m), consisting of 100 000 bags each. A similar closure method will be considered for the western channel.



POSSIBLE GATES STRUCTURE FOR SUDDEN CLOSURE OF EASTERN GAP

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.30

C.7.2.3 Sand closures

Relatively coarse sand is required for this closure method: at least with a D_{50} of 250 micrometer, and preferably more. A deep boring carried out near Char Lakhi did not reveal the existence of such coarse sand. Some other borings are in progress, but it is not expected that the results will be significantly different from the first boring. Coarser sand may be found near Chittagong (in the Karnafuli river), but the long transport distance and the need for stocking and re-handling the sand near the closure site do not offer very bright prospects for an economical closure.

Sand closures have been applied in Europe in the past in situations where the maximum current velocities did not exceed 2.0 to 2.5 m/s. For a sand closure comparable flow patterns will be experienced as for a gradual vertical closure. Computer runs to simulate gradual vertical closures revealed that the maximum current velocities to be expected are substantially higher than 2.0 - 2.5 m/s.

For the above reasons sand closures will not be considered further.

C.7.3 Summary of pre-selected closure methods

The discussion on the various closure methods as presented in Section C.7.2 is summarized in Table C.6. Pre-selected closure methods are indicated by X. Further consideration will be given to two different closure methods for the western channel, two methods for the eastern channel and one each for the two central channels near Char Pir Baksh.

C.8 Structural design of pre-selected closure methods

Each of the pre-selected closure methods will now be discussed in more detail. In principle the closure structure, after having fulfilled its function (stoppage of tidal flow), becomes part of the final dam profile. In some cases it is also possible that the closure structure fulfils the requirements of the cross-dam on its own. Integration of temporary and final functions will be discussed in Section C.10.

Irrespective of the closure method to be selected, bed protection will be necessary over the entire channel width. The length of the bed protection (in the direction of the water flow) depends on the particular location in the channel and on the different construction phases (which may be different for the various closure methods). Bed protection works in the final closure gaps will be discussed in Section C.9.

Table C.6 - Pre-selection of closure methods

Ref. to Section or paragraph	Part of structure and type of closure	Type of closure structure	Construction materials applied	Suitable for closures (width) Western (1500 m)	Eastern (2800 m)	Central West (600 m)	Central East (400 m)	Remarks
C.7.2.1	Substructure	Bed-protection dam or sill	Stones, concrete cubes, gabions, gunny bags	X	X	-	-	
C.7.2.2	Superstructure:							
a-1	grad. vertical	-	Stones, concrete cubes, gabions	X	X	-	-	
a-2	grad. vertical	-	Jetty/cofferdam	-	-	-	X	Closure like Amtali
b-1	grad. horizontal	-	-Stones, concrete	-	-	-	-	Rejected
b-2	grad. horizontal	-	Gunny bags	-	-	X	-	No sill foreseen
b-3	grad. horizontal	Caissons	Concrete	-	X	-	-	
c-1	sudden	Sluice caissons	Concrete/steel	-	-	-	-	Rejected
c-2	sudden	Movable weir	Steel	-	-	-	-	Rejected
c-3	sudden	Closure bund	Gunny bags	X	-	-	-	Closure like Feni
C.7.2.3	gradual	Sand closure	Sand	-	-	-	-	Rejected

C.8.1 Soil mechanics aspects

As already mentioned in Section C.2.6, no information is available on the subsoil conditions in the western channel between Char Pir Baksh and Char Lakhi. However, the similarity between the results of the investigations carried out on the shores of Char Pir Baksh and Char Lakhi, and the results of the subsoil conditions in the remaining part of the alignment, makes it likely that there will not be a great difference in soil composition in these areas. Consequently the soil mechanics aspects considered in this part of the alignment have been based on the results obtained from soil surveys carried out in the remaining part of the alignment.

Using a microcomputer, calculations were carried out to determine the settlement and the stability of a rockfill dam (or a dam of similar materials). The maximum ultimate settlements of the dam and foundation material were found to be in the order of 0.5 m, the greater part of which will occur during the construction period. Differential settlements will be in the order of 0.2 m.

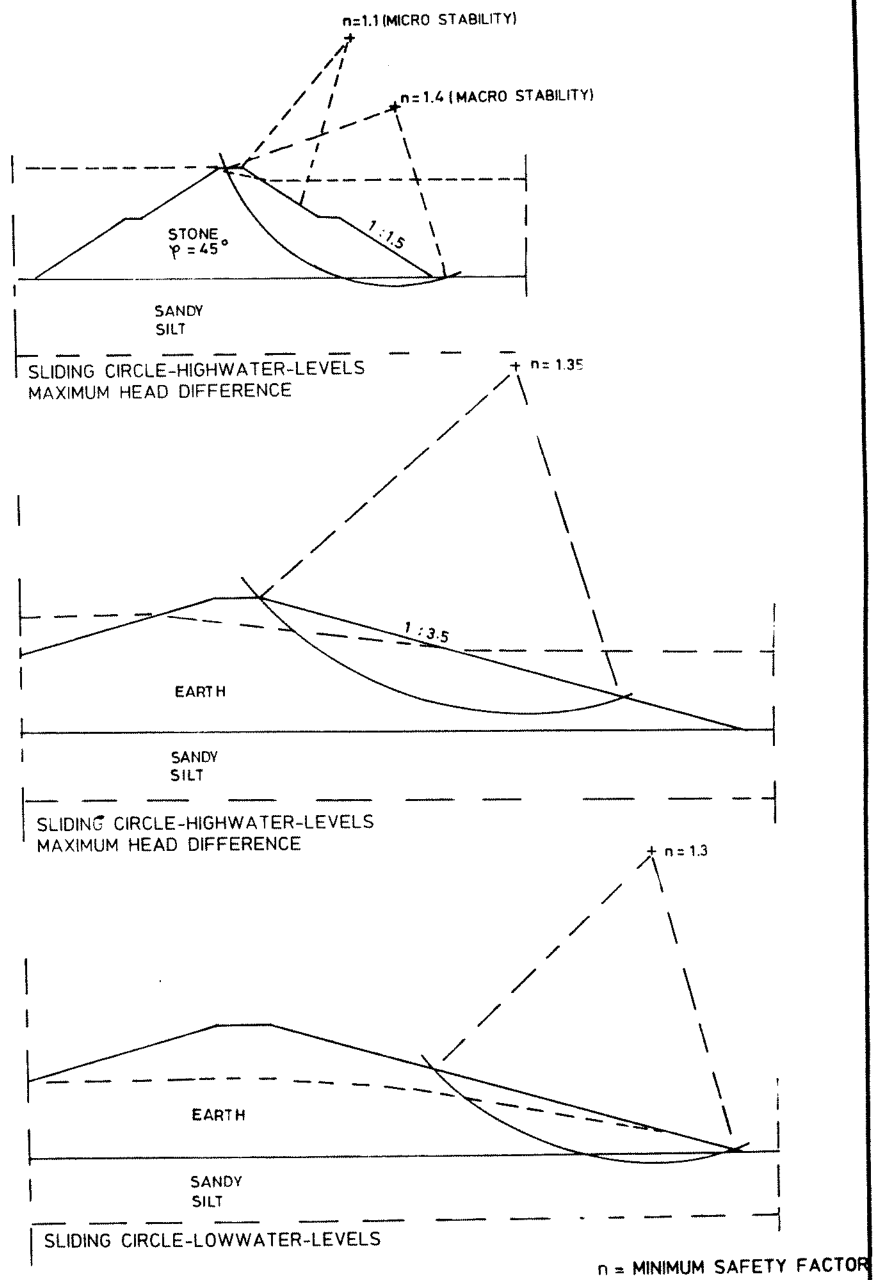
Slope stability and foundation stability were investigated for various loading conditions, occurring in different construction phases and after completion of the dam. Water-levels were derived from hydraulic analyses. The positions of phreatic lines were determined by calculations. The properties of the subsoil and the strength parameters were derived from the soil surveys and the laboratory tests. The properties and the strength parameters of the construction materials were assumed on the basis of hydraulic and geotechnical experience gained from closures and similar structures in the Netherlands.

For computing the steepest allowable slopes the safety factor against sliding was fixed at 1.3, allowing for the variations and uncertainties in the stratification of the subsoil and in the laboratory test results. For the construction phase the safety factor was fixed at 1.1. A seismic coefficient of magnitude 0.04 g has been introduced in the calculations (see Section C.2).

Furthermore, the following parameters were introduced:

Material	Wet density in kN/m^3	Friction angle in degrees	Cohesion in kN/m^2
Sandstone	17.2	40	0
Granite	19.6	40	0
Earthfill	18.0	25	0
Subsoil	18.5	23	3

The results of the stability calculations are summarized in the following paragraphs. (See also Figure C.31).



SLOPE STABILITY-MINIMUM SLIDING CIRCLES

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.31

Macro stability earth dam. Both in the final phase and in the construction phases the most critical loading conditions have been considered: the maximum head difference over the dam at maximum sea level and the condition of the relatively high phreatic line in the dam body at a low sea-level. The calculations indicate that the slopes of the earth dam should not exceed 1:3.5.

Micro stability earth dam. The low plasticity of the fill material renders it susceptible to erosion. However, because of the relatively low head over the dam and the low permeability of the fill material, the seepage will not result in critical exit gradients. Seepage erosion will not be a potential hazard.

Macro stability stone dam. Both in the final phase and in the construction phase the most critical loading condition has been considered: the maximum head difference at maximum sea level. The calculations indicate that the slopes of the stone dam should not exceed 1:1.5 if the internal friction angle of the stone is 45 degrees, and not exceed 1:2 if the internal friction angle of the stone is 40 degrees.

Micro stability stone dam. If a homogeneous stone dam is constructed, in the most critical loading condition a strong current will flow through the very permeable top section of the core. The seepage exit gradients at the downstream side tend to remain below the critical exit gradients for stone material if the slopes do not exceed 1:1.5. If a less permeable crest layer of the dam is chosen due to the necessity to make the crest accessible for (construction) traffic, the exit gradients will become lower and, consequently, the critical exit gradients are not exceeded. It is recommended that more attention be paid to the magnitude of the exit gradients in the final design phase, when more details will be known about the properties of the "hard" materials to be used.

Liquefaction. Prior to checking if the occurrence of flow slides will be a potential hazard (during and after the construction of the closure-dams), it is necessary to investigate if the subsoil is sensitive to liquefaction. Favourable conditions for liquefaction are the presence of thick homogeneous sand layers with low silt proportions and the in situ densities being below the critical densities. The boring results indicate a rather inhomogeneous soil composition at the locations surveyed. The soil consists of alternating sandy silt and silty sand layers. Sand layers with a low silt proportion were found occasionally, but are relatively thin. Laboratory tests carried out on samples selected from these layers of rather low density resulted in critical densities mainly below the in situ densities. Therefore it can be concluded, on the basis of the soil investigations carried out so far, that the probability of liquefaction is not high.

It should be noted, however, that in view of the limited extent of the soil surveys compared to the length of the dam alignment and of the indication, in a few locations, of the occurrence of loose sand layers of limited thickness, the presence of thick, loose sand layers at non-surveyed sections in the alignment cannot be excluded. This will have to be verified in the future by additional soil investigations. In the remainder of this report it will be assumed, however, that flow slides will not occur.

C.8.2 Closure of the western gully

For the closure of the western gully, having at present a width of 1500 m and a maximum depth of PD - 4 m, two alternative closure methods have been studied in detail: a gradual vertical closure method using "hard" materials, and a Feni-type closure (sudden closure) using gunny bags. Both solutions require a substructure, i.e. a sill up to a level of PD - 1 m for the stone closure and PD + 0.0 m for the gunny bag closure. The sill and respective super-structures are discussed in the following subsections. A comparison of the costs of both methods is given in Section C.14.1.2. A comparison of the risks involved for both methods is presented in Chapter C.12.

C.8.2.1 Construction of the sill

A large number of computer runs were made to define the hydraulic conditions which would determine the type of materials for the sill construction. The top level and width of the sill depend on the closure method to be adopted after completion of the sill. Top levels of PD - 1.5 m and PD + 0 m were introduced in the various computer runs, while intermediate construction stages were also investigated.

Due consideration was given to the fact that the tidal ranges (which determine the characteristics of the sill) are larger in the post-monsoon period (lasting till November) than in the winter period (say December till April). It was for instance concluded that intermediate sill configurations (i.e. during construction) which would be acceptable in the winter season, cannot be accepted in the post-monsoon season, unless the diameter of the rock/concrete blocks is increased.Contruction of the upper layers of the sill (say above PD - 2.5 m) should preferably not start before the beginning of December, unless the closure method to be applied sets other requirements.

The head differences which govern the design of the sill are represented in Figure C.32. This graph shows:

- the maximum head difference over the gap (Δh), and
- the overflow height H, for either the N-S on the S-N flow direction, both as a function of the sill crest level (H and Δh do not apply to the same flow situation).

The information contained in the graph can be translated into parameters from which the required minimum weight of elements in the sill can be determined. The first step in the translation process is identifying the flow situations with critical depth on the sill. For situations with subcritical flow over the sill, the discharge is governed by:

- $Q = \mu.b.h.\ \sqrt{(2g.\Delta h)}$,

in which

Q = discharge (m^3/s),
μ = discharge coefficient (1),
b = width of sill (m)
h = water-level difference (upstream and downstream of sill) (m).

For situations with critical depth flow over the sill the discharge is governed by the overflow height (and not by the head difference Δh) as follows:

- $Q = m.b.H^{1.5}$, in which

  m = discharge coefficient (1),
  H = overflow height (m) (Fig. C.32)

Critical depth flow is reached as the downstream water-level, relative to the sill crest level, is less than 2/3 of the overflow height H. A further lowering of the downstream water-level increases the current velocities on the downstream part (slope) of the sill, without an increase of the discharge.

For each closing stage there is a combination of overflow height H and downstream water depth h that determines the design situation on the crest of the sill. In these situations the elements at the shoulders of the sill are most vulnerable to current attack. It is at those locations that the loss of stability of sill elements will start, unless they have sufficient weight. Such situations occur for the low water flow direction (S-N) for low sill levels and for high water flow direction (N-S) for high sill levels.

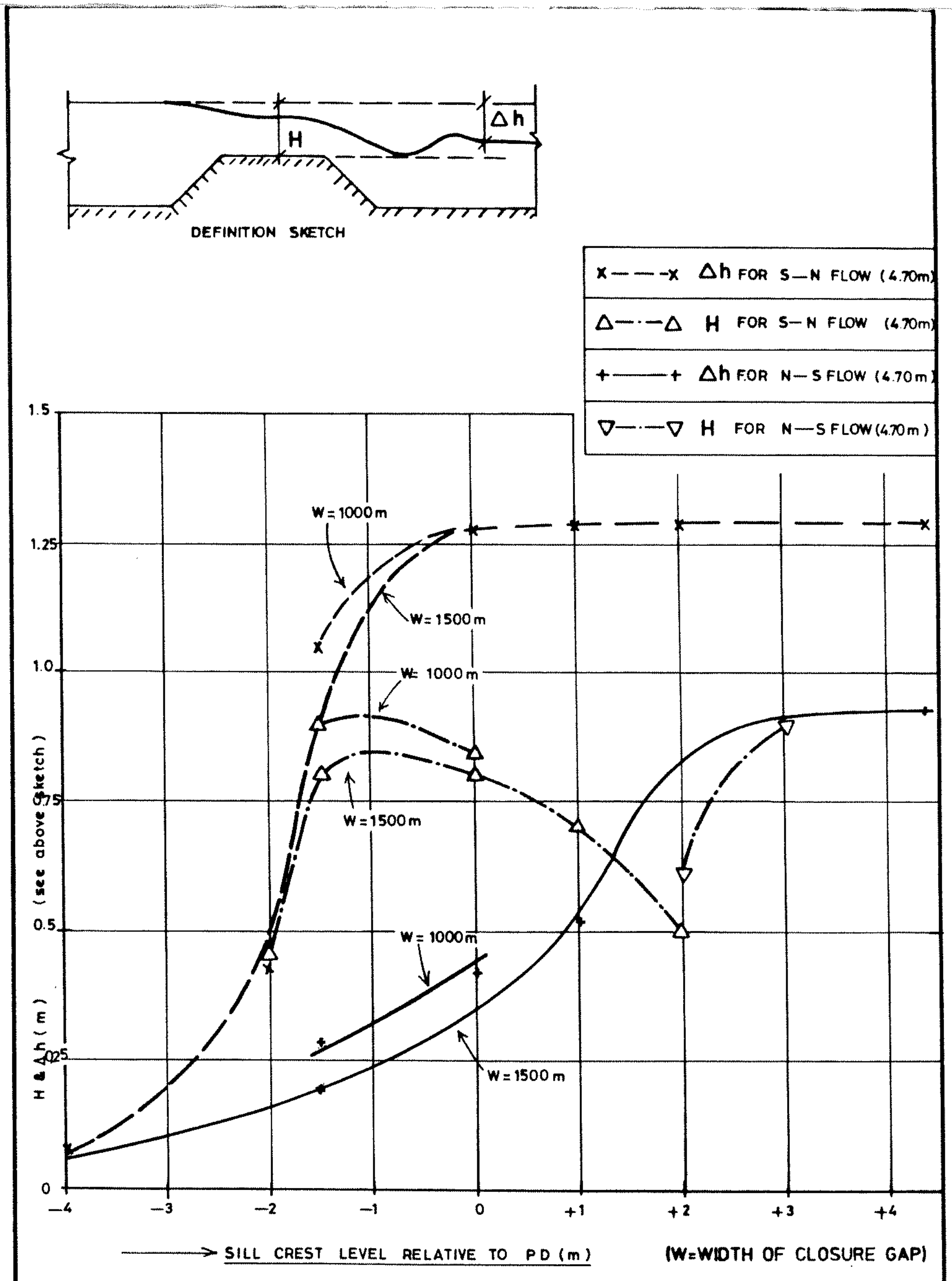
The actual combination of the overflow height H and the downstream water-level h_d can be translated into a design-governing overflow height H corresponding to a downstream water-level equal to the top of the sill ($h_d = 0$). This translation is based on extensive model investigations carried out in the past by the Delft Hydraulics Laboratory. As a result of this translation the design-governing overflow height H can be directly related to the parameter ΔD from which the required minimum weight of the elements can be determined according to:

$$H_d = c.\Delta.D$$

in which:

H_d = design-governing overflow height (m)
c = coefficient, which is a function of sill crest width and the side slope angles; for the present sill configuration c = 1.75.
Δ = relative density of closure element under water = $(\rho s-\rho)/\rho w$
ρs = density of closure element (kg/m^3)
ρw = density of (sea) water (kg/m^3)
D = Dn "diameter" of the closure element (taken as the third root of the volume) (m)

The minimum "Δ.D" for some characteristic situations in the graph, and their subsequent translation into the minimum weight of a sill element, is given in Table C.7. (This weight also depends on the density of the material itself.)



HEAD DIFFERENCES DURING CLOSURE AT THE WESTERN CLOSURE-GAP

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.32

Table C.7 - Minimum weights of sill elements, western closure

Sill or dam crest level (m rel.to PD)	H (m)	H*1.15 (m)	Δ.D (m)	W1 (kg) (Δ=1.17)	W2 (kg) (Δ=1.57)	W3 (kg) (Δ=0.75)	W4 (kg) (Δ=0.55)
Tidal range = 4.70 m							
-2.0	0.45	0.52	0.30	37	18	115	260
-1.5	0.80	0.92	0.53	205	100	635[1]	1432
-1.0	0.85	0.98	0.56	241	118	750[1]	1689
+3.0	0.90	1.04	0.59	282	138	876[1]	1975
Tidal range = 5.60 m							
-2.0	0.60	0.69	0.39	81	40	253[1]	570
-1.5	1.10	1.27	0.73	534	261	1660[1]	3741
-1.0	1.15	1.32	0.75	579	283	1800[1]	4057
+3.0	1.20	1.38	0.79	677	331	2104[1]	4741

Notes:
W1..4 = weights of sill elements.
W1 = for sandstone or concrete (ρ = 2200 kg/m^3).
W2 = for limestone or granite (ρ = 2600 kg/m^3).
W3 = for "clay bag" (ρ = 1800 kg/m^3).
W4 = for brick gabion (ρ = 1600 kg/m^3).

[1] Clay bags with such weight are of course nonexistent. Clay bags have however been included in the table to illustrate that their use as single element in the sill is not possible on purely technical grounds.

The overflow heights H represented in the graph are valid for a flat sill over the entire width of the gap. In practice, however, irregularities of one block diameter can be expected. Owing to these irregularities a somewhat (15 %) larger overflow height will be experienced, which has been used in Table C.7 for determining the "stone" diameters.

The weights refer in principle to elements on the shoulders and slopes of the sill. The elements in the central part could in principle be lighter, but the central part may be exposed to maximum current attack during intermediate construction phases, when materials on the shoulders have not yet been placed.

The following principal conclusions can be drawn:

(a) if the sill reaches full height (PD + 0 m) at the end of the post-monsoon season for a closure gap of 1500 m, then sill elements with a weight of 677 kg (assuming sandstone or concrete Dn = 0.68 m) will be required.
(b) if the sill reaches full height in the winter season (assuming a start of construction by early December) for a closure gap of 1500 m, then sill elements with a weight of 282 kg (Dn = 0.50 m) are required.

The above figures are all based on the calculations performed with the mathematical model. The influence of wind was not included. If southerly winds lead to unequal wind setup in the Sandwip and Hatia channels, then the head differences over the sill may increase. This requires heavier sill elements in principle. On the other hand it should be considered that the sill is very wide. Repairing occasional light damage during construction may be cheaper than using heavier elements in the entire sill. Detailed consideration needs to be given to this aspect in the final design stage, both from construction and contractual point of view.

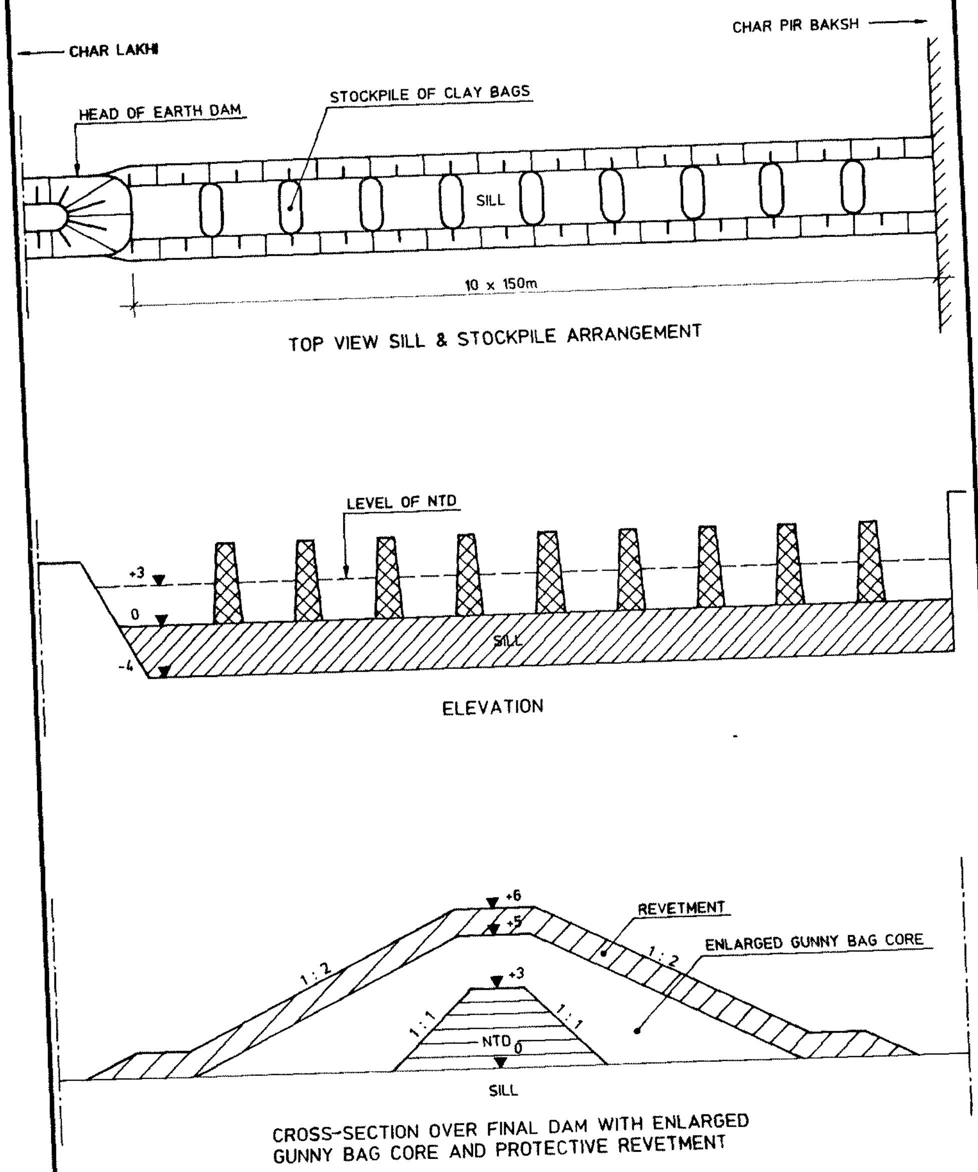
C.8.2.2 Sudden closure with gunny bags

This method requires a sill that will become dry during a considerable period during neap tide (best period is January/February). The top level of the sill has been determined at PD + 0 m. Stockpiles of clay bags will have to be made on the sill at regular intervals, say 150 m (Fig. C.33). On the closure day these stockpiles should be "turned 90 degrees" when the sill has become dry. This operation should be completed before the next high water, so that roughly six hours will be available for the closure operation.

This method was successfully used for the Feni dam. For the closure of the western channel of the Sandwip cross-dam the situation is logistically more difficult, as all gunny bags will have to be transported to the stockpiles by barges. This was not the case in Feni, where the majority of the bags could be carried by labourers over the shoals during low water (the width of the gully in the Feni river was only 250 m: substantially less than for the western closure). Consideration should also be given to the stability of the gunny bags in the stockpiles, where they will be subjected to large current velocities parallel to the axis of the stockpiles.

The closure dam should reach a height of PD + 2.5 to + 3.0 m on the closure day, and should thereafter be reinforced to withstand the first spring tide after the closure. This level has tentatively been fixed at PD + 4.5 m. In subsequent stages the dam should be further reinforced and protected, which could be done in various ways. The work required after the closure could be completed before the pre-monsoon cyclones and the monsoon season. Figure C.33 gives a cross-section of a possible final dam at the closure-gap location, the core of which is formed by gunny bags. A rather strong revetment is required over the enlarged gunny bag core.

While the closure method itself is labour-intensive, this cannot be said for the works preceding the closure. As already mentioned, a sill level at PD + 0 m is required. In view of the scheduled time of the closure and the required period for stockpiling of the gunny bags, the sill should be completed in December, implying that construction has to be started too early to take advantage of the favourable winter tides. Moreover, a sill at level PD + 0 m cannot be completed entirely within an acceptable time limit with the marine equipment necessary for the construction of the lower layers of the sill. Therefore the upper layer will have to be made (at least) partly with land-based equipment.



-WESTERN CLOSURE-ARRANGEMENT OF STOCKPILES FOR SUDDEN GUNNY BAG CLOSURE
-CROSS-SECTION OVER POSSIBLE FINAL DAM

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.33

C.8.2.3 Gradual closure with rock/concrete blocks

This method emerged as a potentially very attractive closure method for the eastern closure (see Section C.8.3), but it can also be adopted for the western closure.

The gradual stone closure is basically a vertical closure method; a dam of "hard" materials is constructed in layers of moderate thickness (say 2 m) on top of the sill. The top level of the first layer of the dam could be constructed with land-based equipment at PD + 1 m, assuming that a level of PD - 1 m will be reached with marine equipment. The use of a layer of 2 m thickness on top of the sill crest at PD - 1.0 m leads to a higher overflow height (H = 1.05 m, tidal range = 4.70 m) than represented in Figure C.32. However, the critical stage (the last part of the layer) in this operation can be planned during a neap tide period.

Approximately 100 000 m^3 of "hard" materials is required for the rockfill dam on top of the sill, which could be constructed within one month, assuming that only one stockpile (at Char Pir Baksh) will be created. This implies that the work on the sill could be postponed till December/ January, so that lighter sill elements are required.

The closure of the gap with one layer with a thickness of 6 m (-1.0 to +5 m) will introduce strong three-dimensional effects in the closure-gap. (In fact this method is a horizontal closure.) Although it is difficult to determine the 3D-effects without the aid of model investigations, a rough estimate indicates that the diameter of the stones on the bed protection should be 70 % larger. This also means a 70 % larger quantity of stones on large parts of the bed protection. The costs involved are rather high and therefore this alternative has not been worked out in more detail. Also the diameter of the blocks to be used for the closure should be larger, but this will not lead to an increase of the quantities involved.

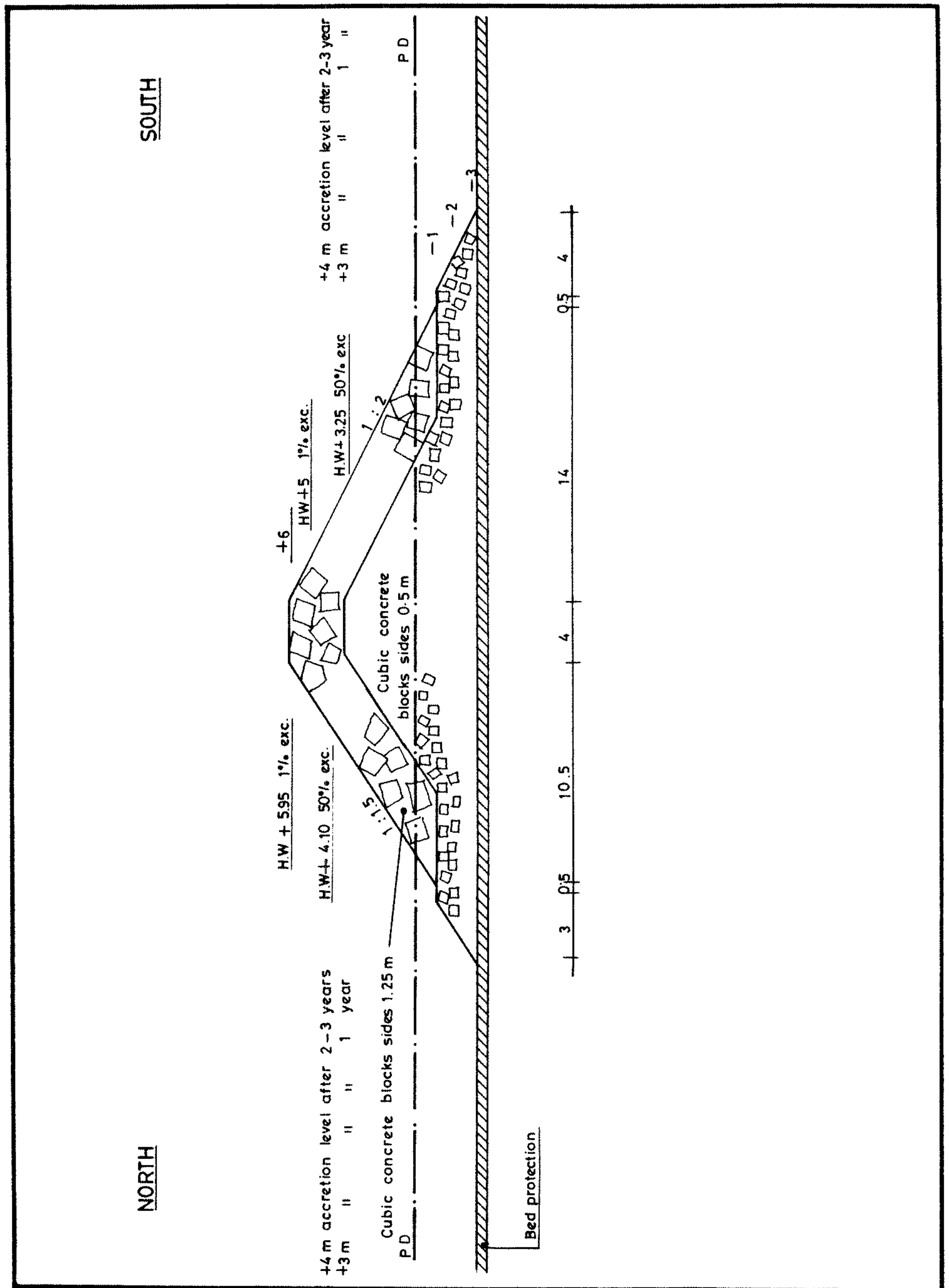
The rockfill dam on top of the sill will serve the purpose of stopping the flow between Sandwip and Hatia channels. A careful selection of the stone gradings (rock, concrete blocks, etc.) to be used in different locations in the body of the dam, could render it suitable to act as an overtoppable break-water. In that case the volume of the rockfill dam has to be increased, but the larger volume would eliminate the need for a surrounding or adjoining earth body with protective revetments (see also Section C.11). A cross-section over a rockfill dam without surrounding earth body is given in Figure C.34.

C.8.3 Closure of the eastern gully

For the closure of the eastern gully (2800 m wide, maximum depth PD - 8m) two alternative closure methods were studied in detail:

- a gradual vertical closure method using "hard" materials, and
- a gradual horizontal method using caissons.

A sill is required for both methods.



CROSS-SECTION OF THE CLOSURE DAM
WESTERN CHANNEL FROM km 3.75-km 5.25

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.34

A Feni-type solution for the closure of the eastern gully is not suitable. This solution will lead to larger volumes, higher costs, greater risks and a longer construction period than using 'hard' materials. The arguments to go for a solution using 'hard' materials as discussed for closing the western gully are even more valid for the much wider eastern closure gap.

C.8.3.1 Construction of the sill

The design for the sill was conducted along similar lines as described for the western closure. Top levels up to PD - 1 m and PD + 0 m were introduced in the computer runs. The head differences which govern the design of the sill are presented in Figure C.35. For interpretation of the graph reference is made to Section C.8.2.1.

A translation of the several situations into "Δ.D" and the weights of sill elements is given in Table C.8.

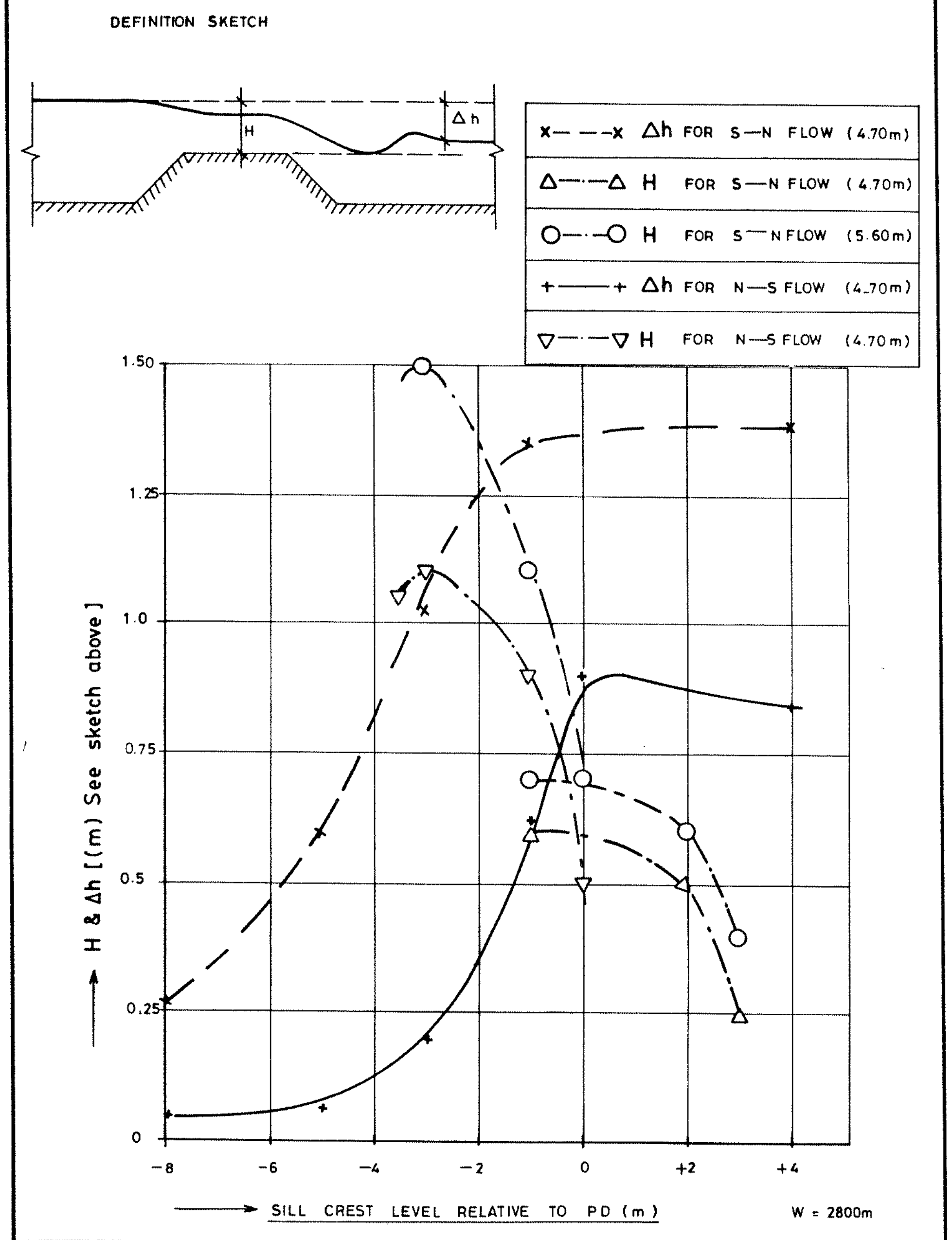
The block or stone weights in the table are strictly speaking only sufficient for a further gradual vertical closure in restricted layers in a closure-gap of 2800 m, and not for a caisson closure. Preliminary calculations with the SEFLO-model lead to the conclusion that a further narrowing of the gap to 2100 m will hardly increase the dimensions of the blocks. A larger width of the closure-gap (4200 m) leads to a small reduction in the maximum block weight.

At a sill crest level of PD - 3.0 m the effects of irregularities in the sill crest level (± 0.75 m) do not lead to larger overflow heights. For a crest level at PD - 1.0 m there is an increase of H to 1.5 m. This is rather a large increase, but the required block size does not exceed the size at an (intermediate) crest level at PD - 3.0 m.

For a horizontal closure with closed caissons, special attention has to be given to the three-dimensional effects caused by the large discontinuity of the cross-section at the front of the caissons already placed. The use of heavier blocks has to be expected.

C.8.3.2 Gradual vertical closure using "hard" materials

The principles of a gradual stone closure have been described in Subsection C.8.2.3. With adequate planning (making use of neap tide periods) and a well-balanced dumping method it is possible to make a block structure in fewer layers than in the western gap. For the final design this must be worked out in more detail.



HEAD DIFFERENCES DURING CLOSURE AT THE EASTERN CLOSURE-GAP

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.35

Table C.8 - Minimum weights of sill elements, eastern closure

Sill or dam crest level (m rel.to PD)	H (m)	Δ.D (m)	W1 (kg) (Δ=1.17)	W2 (kg) (Δ=1.57)	W3 (kg) (Δ=0.75)
Tidal range = 4.70 m					
-5	-	0.28	30	15	221
-3	1.10	0.63	343	168	2405
-1	0.90	0.52	193	94	1352
0	0.60	0.34	54	26	378
+2	0.50	0.28	30	15	211
Tidal range = 5.60 m					
-5	-	0.38	75	37	528
-3	1.50	0.86	873	427	6117
-1	1.10	0.63	343	168	2405
0	0.70	0.40	88	43	615
+2	0.60	0.34	54	26	378

Notes:
W1..3 = weights of closure unit,
W1 = for sandstone/concrete (ρ = 2200 kg/m^3).
W2 = for limestone/granite (ρ = 2600 kg/m^3).
W3 = for brick gabion (ρ = 1600 kg/m^3).

Approx. 340 000 t of rock or other hard materials is required for a rockfill/concrete dam of 2800 m length. While the prospects of finding a large quarry site in the Chittagong Hill Tracts are not encouraging, it has been concluded that replacement of local rock by precast concrete blocks or imported rock (for instance from India or Malaysia) does not rule out the gradual stone closure as an attractive solution. On the contrary, the gradual stone (or concrete block) closure allows the closure to be completed fast, at minimal risks and (comparatively) cheaply.

If the proper stone gradings are selected for the closure-dam, no additional works are necessary after the closure. (The final crest level in the eastern closure is lower than for the western closure, so that the final profiles are not identical.)

The stability of a rockfill/concrete dam against wave attack etc. is discussed in Section C.11.

C.8.3.3 Gradual horizontal closure with caissons

This closure method has been successfully applied on a number of occasions in the Netherlands. The principle is that a long closure-gap, which has been made rather shallow by means of a sill, is closed horizontally by sinking caissons. Because of the height of the sill the current velocity on the sill cannot pass a certain value and the closure-gap can be gradually narrowed in consecutive days by placing caissons during each high tide.

For this closure method the sill in the eastern gully has been designed at a level of PD - 0.5 m. This enables the preparation of the sill (levelling and trimming) to proceed during low waters. It was concluded that caissons can only be placed when astronomical high water levels are PD 3.10 m or higher, in view of: the keel clearance and draft of the caissons; the unevenness to be expected in the top layer of the sill; the movement of the caissons during the placing operations; the time required around high slack water for manoeuvring and sinking.
This water-level is reached in February for only 10 % of the tides. For January this figure is even less. Placing beyond that period would be very risky.

The costs of this closure method were found to be higher than those of a gradual vertical closure using hard materials. Other disadvantages are:

- as part of the final dam the caissons are vulnerable to heavy wave attack (vertical facing);
- preparation of the sill is time-consuming and requires an early start of the construction of the sill, i.e. in the post-monsoon period. This in turn means the use of heavier elements in the sill.

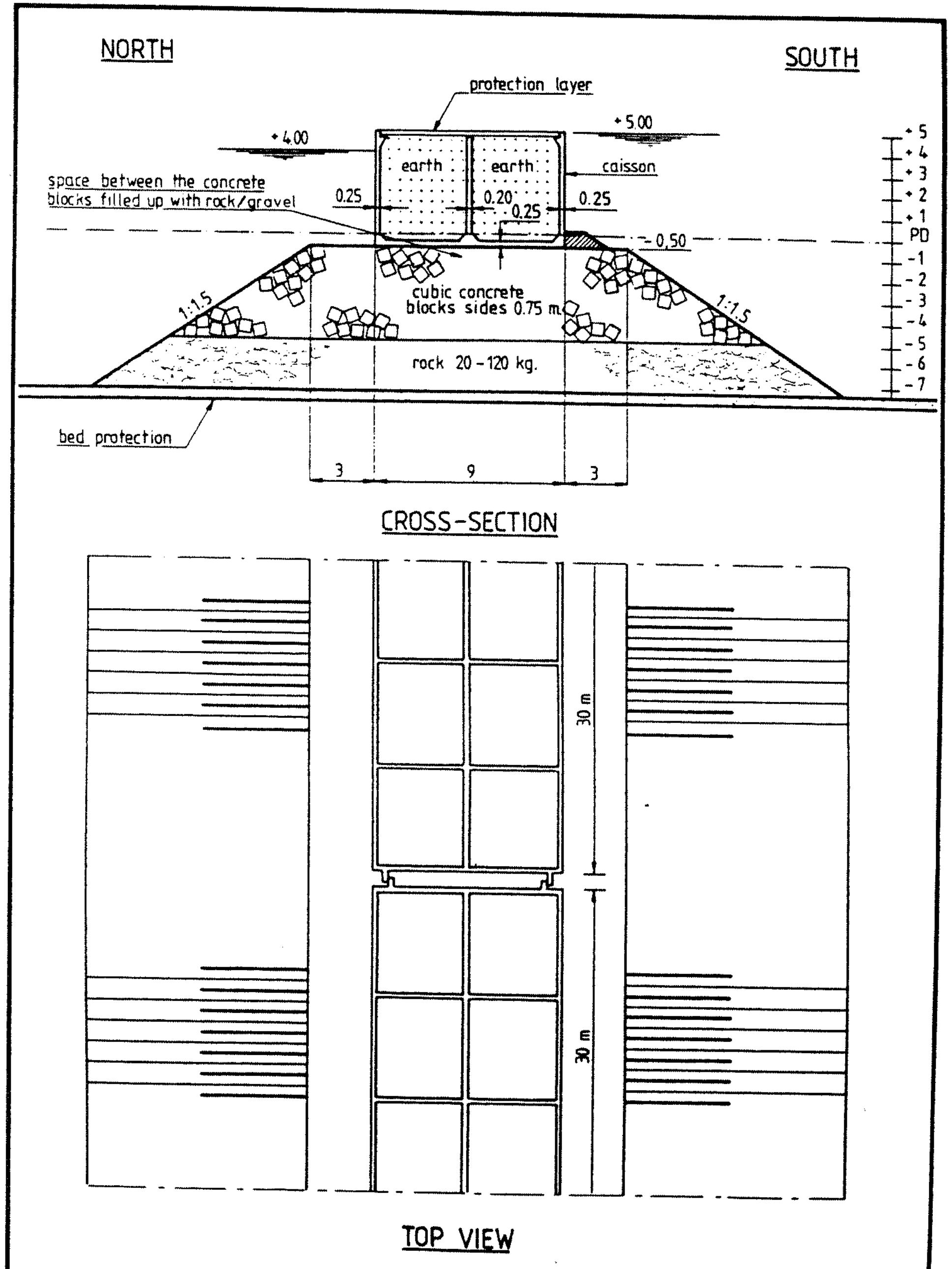
Sketches of a possible caisson configuration are given in Figure C.36.

C.8.4 Closure of the central channels near Char Pir Baksh

As stated in Section C.7.3, only one closure method will be considered for each closure-gap. The reason is the relatively easy closure operation which renders it possible to use methods which have been tested in Bangladesh and can be carried out with local materials and (predominantly) labour.

C.8.4.1 The eastern central gully

As stated in Section C.7.2.2, paragraph a-2, this gully can be closed with jetty/cofferdam structure built up of gunny bags. This gradual type of closure does not require a level which runs dry for a considerable time at low water. The method involves gunny bags being dumped within the confinement of a timber jetty, from which the bags cannot escape. The bags and the jetty together form a cofferdam. The substructure for this closure can probably be limited to a bed protection which is a ballasted with clay-filled gunny bags or brick in gunny bags.



EASTERN CHANNEL-CAISSON CLOSURE (ON HIGH SILL)

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.36

During the closure operation (a few consecutive days around a winter neap tide) gunny bags will be carried by labourers over walkways (provided on the jetty structure) and dumped in the water. Following the closure, sufficient height has to be reached in the subsequent days to withstand the first spring tides after the closure operation. When this has been achieved the closure-dam has to be reinforced further to withstand the higher monsoon water-levels. The level of this earth dam should match the level of the other earth dam sections on Char Pir Baksh.

C.8.4.2 The western central gully

This gully has a present width of 600 m and a maximum depth of PD + 0 m. This rather shallow depth permits application of a gradual horizontal closure by working from both banks. It is essential, however, that a bed protection be constructed first over the full width of the gully. Moreover, the clay bags to be used for the closure bund must be prepared in advance and stacked at both banks near the axis of closure. The closure must take place on a few consecutive days around a (winter) neap tide. The earth dam to be built around the closure bund should match other earth dam sections at Char Pir Baksh.

C.9 Bed protection in the final closure-gaps

The previous section defined the sizes of rock required for the closures under the limiting conditions. The bed protection will be subjected to the same conditions. Therefore these conditions were also used for the design of the bed protection mattresses and ballast as well as the timing and methods for their placement.

C.9.1 Construction

Initially, the bed protection in any of the the gullies will be subjected to moderate flow attack, but when the construction of the sill has started the situation will change drastically. The bed protection should be constructed as a mattress consisting of two layers of geotextile, a non-woven type with openings smaller than the bed material and on top of it a strong woven type. A framework of bamboo will be fixed to the geotextiles to provide the necessary buoyancy for the marine operations. The mattresses have to be sunk around slack water, and covered with ballast; the weight of the individual elements must be sufficient to withstand the current forces. The length should be such that the mattress is not undermined at the edges. It should be noted that a fabric of natural materials, such as reed, goalpata leaves or bamboo, cannot be applied for various reasons: insufficient strength (for gully mattresses), insufficient sand-tightness, inavailability of goalpata leaves in the season when they are required, and long assemby time for shoal mattresses.

C.9.2 Length of the bed protection

The length of the bed protection is directly related to the scour phenomenon. Scour has to be expected at every location where the

sediment carried with the current is not in equilibrium with the sediment transport capacity of the water. Such a situation occurs in the closure-gap, where the current velocities are much higher than in the channel section in front of the gap. Downstream of the gap the transport capacity remains high due to turbulence created by the deceleration of the current. The formation of a scour hole downstream cannot be prevented by application of bed protection works. The main function of the bed protection is to create and to maintain an adequate distance between the scour hole and the sill or dam under construction, so that the scour hole cannot endanger the stability of the sill or dam.

In Section C.8.1 it has been stated that the occurrence of flow slides is not likely and that soil slides will only occur if the underwater slopes exceed 1:2, which can be regarded as a steep slope. The length of the bed protection should be such that steep slopes are prevented, so that erosion under the mattress is not likely to occur. To meet this requirement the current from the sill must have horizontal "support" the bed protection. In other words, the bed protection must be so long that the eddy downstream of the sill is situated entirely over the bed protection (Figure C.37). In that case the spreading of the current in the scour hole will be minimal. This will result in a slope of the scour hole which is as flat as possible. For the spreading of the current an angle of 1:7 has been taken and the horizontal part of the bed protection must have a length of at least twice the water depth for an effective horizontal support of the current.

When the level of the sill is raised, the current velocity over the sill and the rate of turbulence downstream of the sill will increase. Consequently there will be an increase in the scouring capacity of the current. On the other hand, raising of the sill will also cause a decrease of the discharge. These phenomena combined lead to a maximum scour capacity at a sill level of 0.6 - 0.7 times the water depth.

Based on the foregoing considerations the length of the bed protection follows from:

$$l = 7 \times 0.7\,h + 2\,h = 6.9\,h$$

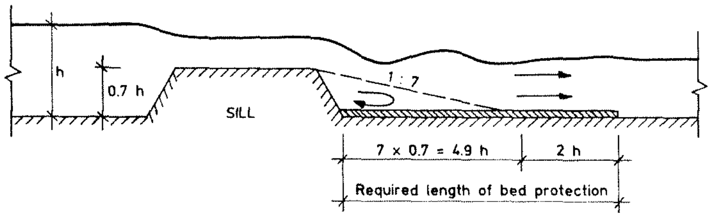
in which:

l = length of bed protection measured from the toe of the sill or dam under construction
h = water depth = difference between the slackwater-level for the current direction concerned and the local bed level.

A minimum length of 30 m is considered safe and practical.

The above considerations have led to the design of the bed protection (associated with stone closures) presented in Figure C.38.

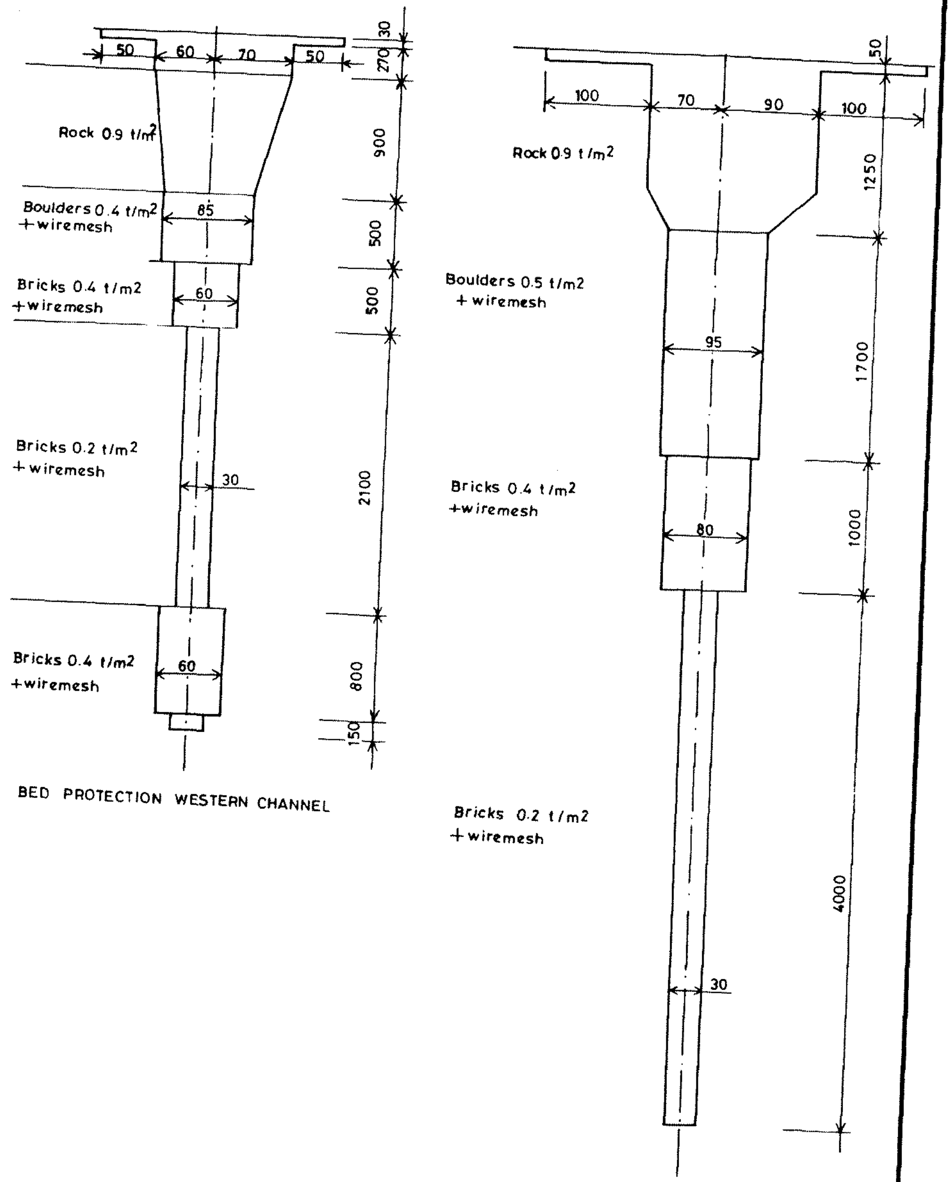
It is further necessary to construct the edge of the bed protection so that the mattress can flexibly follow bed irregularities without losing the stones which have been dumped on it. Moreover, some model tests are recommended to verify the above.



SCHEMATIC CROSS-SECTION OVER SILL AND BED PROTECTION

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG C.37



DIMENSIONS IN M.

LAY-OUT OF BED PROTECTION WORKS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG C.38

For a gunny bag closure in the western gap the formation of stockpiles will a further narrow the gap and more turbulence downstream can be expected. This will result in a a somewhat steeper slope of the scour hole and a higher chance of undermining the edge, so longer mattresses may be required.

The bed protection in the central channels has not yet been determined for lack of data. In view of the absence of sills the bed protection can probably be relatively short.

C.9.3 The ballast on the mattresses

The individual units of the ballast on the mattresses should be stable under the current attack during construction of the sill and subsequent closure works. The sill height influences the maximum velocities over the bed protection. The closure method and season will determine the stone size required. When the sill is low in comparison with the depth, the $\Delta.D$ of the stones of the bed protection will be the same as the $\Delta.D$ necessary for the construction of the sill.

For stability reasons the ballast layer must have a thickness of twice the average stone size. In that case the upper layer has sufficient grip on the lower layer to use the applicable stability criterium. To minimize the total volume of the ballast, rock with a density of ρs = 2600 kg/m^3 has been assumed. A framework of bamboo protruding above the first layer will have a favourable effect on the stability of the upper layer.

Western closure (final gap)

For a stone closure, the largest current velocities in the direction from Hatia channel to Sandwip channel occur for a sill crest level at PD - 2.0 m. The sill is low and the water depth relatively small and therefore the $\Delta.D$ used for the sill has to be used also for the bed protection.

$\Delta.D$ sill = 0.56 m = $\Delta.D$ bed protection

Δ rock = 1.57, thus D bed protection = 0.36 m

Total thickness of ballast = 2 x 0.36 m = 0.72 m

For the current direction from Sandwip channel to Hatia channel the highest velocities occur for sill crest levels at PD + 1.0 m and PD + 2.0 m. For the deeper parts of the closure gap the current from the sill crest can spread and the permeable sill prevents the occurrence of a diving water jet. The necessary stone size has been calculated as D = 0.2 m. However, in the shallower part of the closure gap the situation is less favourable and in that part the same rock diameter as used for the sill is necessary. As an average for the southern part a stone size D = 0.3 m can be used, which means an average thickness of the ballast of 0.6 m.

For a gunny bag closure the velocities will be somewhat higher (due to the stockpiles), and 3-dimensional effects will occur; consequently the stone size of the bed protection has to be increased (roughly 15 %). The time required to build up the stockpiles is expected to be considerable. Therefore the construction of the sill has to be started earlier (in the less favourable post-monsoon season). As a result the stone size for the bed protection has to be increased by 35 %. The layer thickness will increase correspondingly. The stages after the construction of the sill will be less critical in that case.

Eastern closure (final gap)

For a stone closure the largest current velocities in the direction from Hatia channel to Sandwip channel occur for a sill crest level at PD - 3.0 m. In the shallower part and in the transition zone to the deeper part the diameter of the bed protection material should be the same as the diameter used for the sill material. This leads to stones with an average diameter of 0.55 m (0.86/1.57) and a total thickness of 1.10 m. In the deeper part, the current attack is less and a stone size of D = 0.27 m has been determined, which implies a total thickness of 0.54 m.

For the current direction from Sandwip channel to Hatia channel the largest velocities occur for a sill crest level at PD - 1.0 m. In the shallower parts south of the sill the same $\Delta.D$ is necessary as calculated for the sill material. This means an average diameter of 0.26 m and a total thickness of 0.52 m. Although the diameter of the stones in the deeper part may be somewhat smaller than 0.26 m it is recommended that the same diameter and thickness be used as for the rest of the closure-gap.

For a caisson closure, with three-dimensional effects in the closure-gap, the current attack on the bed protection will be stronger. For the current direction from Hatia channel to Sandwip channel this attack is not critical in view of the high crest level of the sill on which the caissons will be placed. For the current direction from Sandwip channel to Hatia channel the 3-dimensional effects lead to larger stones. A rough estimate indicates that an increase of 70 % over a large part of the closure-gap width must be expected.

C.10 Narrowing of the western and eastern channels

The Sandwip cross-dam is located for a considerable part on shoals which run dry for a few hours during each tide. In the present morphological situation the cross-dam can be considered to run over the shoals over distances of 3750 m (western channel) and 5200 m (eastern channel). In the western channel a second gully can be distinguished near Char Lakhi. Though the dimensions cannot be neglected, the dam through this gully has not been considered as a closure-dam. During construction the water-levels at both sides of this dam section will be almost equal, because the area of the shoal and main gully (near Char Pir Baksh) are still open at that time. A longitudinal section of the present system of shoals and gullies has already been given in Figure C.12 (Section C.2).

As will be explained in Section C.10.2 the dam sections on the shoals will be made of earth. As working conditions in Bangladesh do not permit large-scale earthwork to be done outside the dry season, it has to be assumed that the dam sections on the shoals will be constructed in the period December-February. During this period not much wind is to be expected and wave attack will be minor.

Subsoil conditions in the area to be traversed by the cross-dam have been described in Section C.2.

C.10.1 Design considerations for dam or other structure on the shoals

When designing the cross-dam on the shoals the functions of this dam should be recalled. These functions are:

- In the initial stages, when the western and eastern channels have not yet been closed, the dam will function merely as a structure preventing tidal flow across the shoals. Depending on the crest level of the dam (discussed in Section C.11) water may occasionally flow over the dam and waves may break on its crest.
- As soon as sufficient land has accreted on both sides of the dam, it will have lost its function completely. This latter may happen within 10 years after completion of the cross-dam. Bearing in mind this temporary character, some sketch designs have been made of structures which would serve the purposes described (Figures C.39a and b).

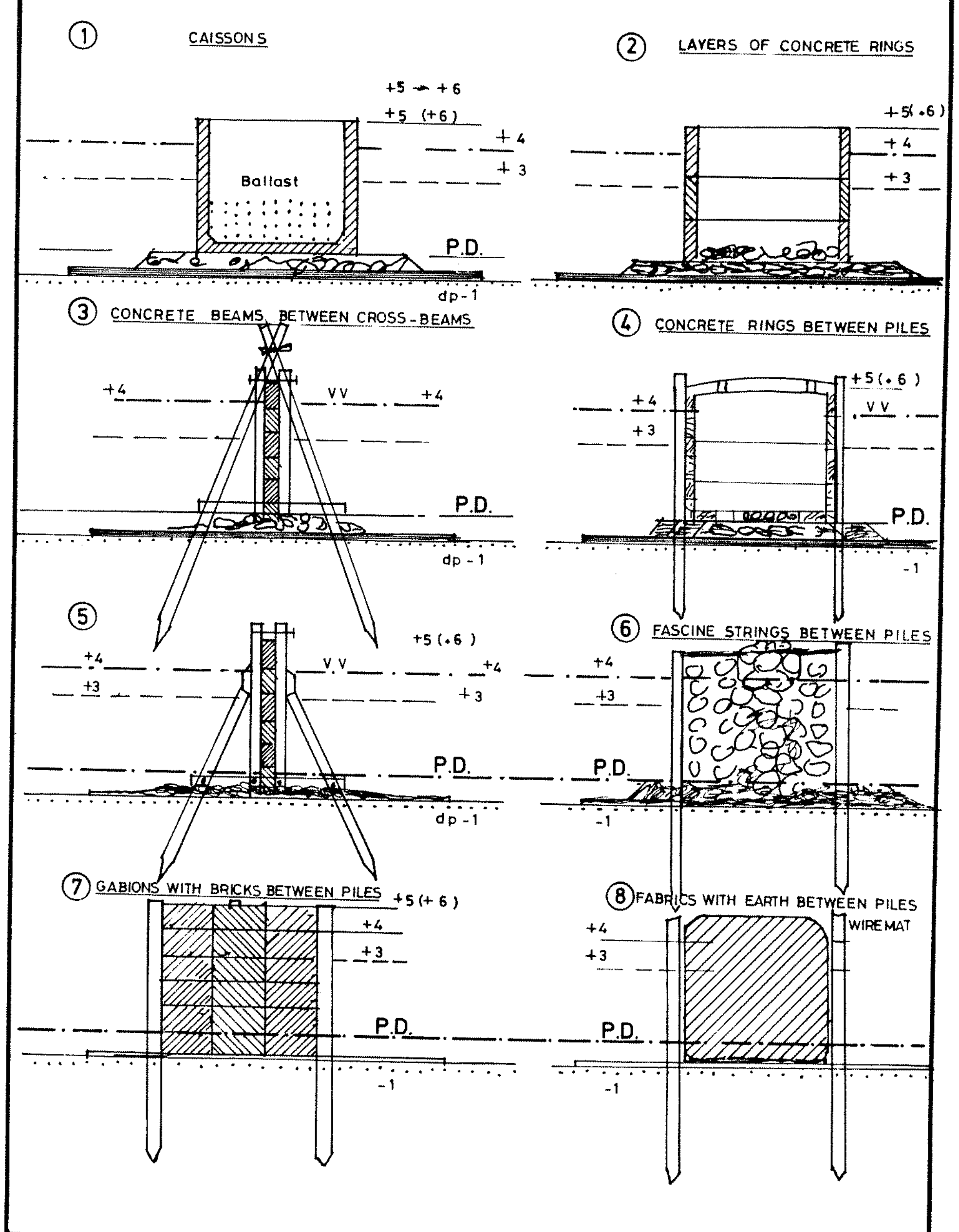
The following aspects of the structure have been investigated:

- cost of supply of materials and construction;
- stability against (tidal) water-level differences, overflow and wave attack (combined loading from one side is 20-30 kN/m^2);
- lifetime;
- availability of materials in Bangladesh;
- permeability in view of piping;
- construction time;
- risks during construction.

Assessment of the possible solutions indicated in Figures C.39a and C.39b led to the conclusion that a dam profile of earth with a protective revetment will provide the most economical, stable and reliable solution.

C.10.2 Construction aspects

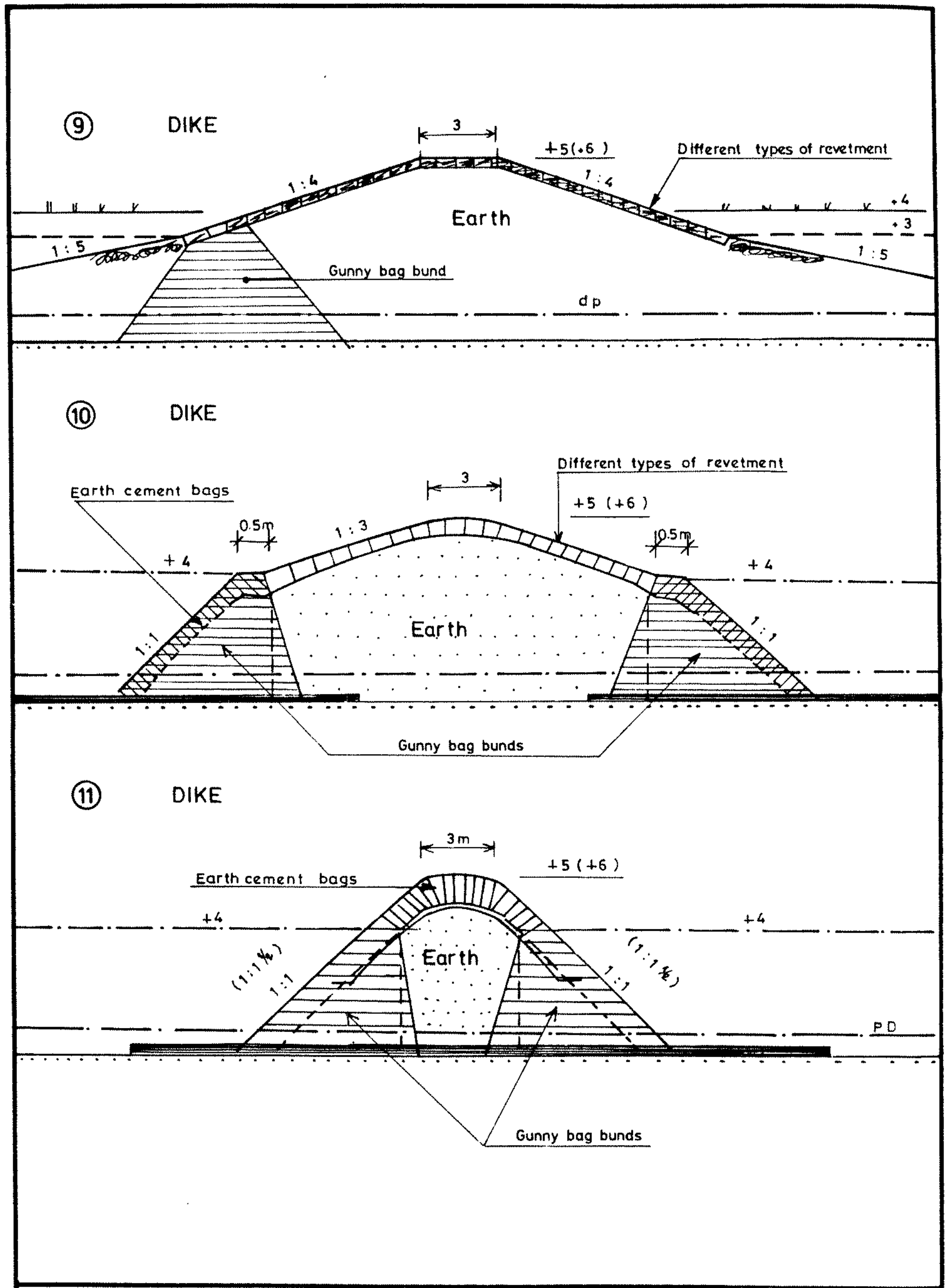
Current velocities across the shoals will increase around the heads of the earth dam sections built forward from Char Lakhi and Char Pir Baksh. In view of the easily eroding subsoil it is necessary to place a bed protection along the full alignment of the dam sections on the shoals. This bed protection can be placed during low water well in advance of dam construction. Its width varies between 30 and 60 m. Ballasting can be done with bricks, while wiremesh has to be fixed to the bed protection to prevent the bricks from being removed by the current.



POSSIBLE STRUCTURES FOR DAM ON SHOALS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.39a



POSSIBLE STRUCTURES FOR DAM ON SHOALS

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.39b

It will also be necessary to prevent erosion of the earth dam itself during construction. This can be done by first building a low bund from earth-filled gunny bags. Placing of earthfill would take place in the "shadow" of this bund. Only one bund will probably be required in view of the prevailing current patterns (Figure C.39b).

Dam sections on the shoals will require approx. 500 000 m^3 (western channel) and 600 000 m^3 (eastern channel) of earthfill. The earthworks for each dam-section should preferably be completed in one dry season. A longer construction period (two years instead of one) implies that the (light) bed protection will have to function longer, that increased scour can be expected in the (narrowed) tidal channels and that the closures will be realised later (probably one full year).

The magnitude of the earthworks to be carried out and the impossibility to exploit borrow pits along the alignment of the dam (shoals are only dry during a few hours of each tide and any excavated borrow pit will immediately fill up with water) dictates the use of earthmoving equipment. The long haulage distance for the earth (on average 3 km) and the restricted workfront preclude the use of local trucks, which are normally not of the tipping type and have to be unloaded manually. Therefore efficient dump trucks and excavators or loaders will have to be used to achieve the required high earthmoving production. During low water the situation is most favourable for advancing the head of the dam on the shoals.

Earthwork sections which are (partly) complete, but on which no revetment has been placed yet, must be temporarily protected against erosion by waves and currents. This could be done, for instance, by placing bamboo mats ballasted with gunny bags.

It is estimated that reasonable progress with the earthworks can be made if the groundlevel (on the shoals) is PD + 0 or higher. This implies that the last 1000 m of the dam on the shoals of the eastern channel may have to be constructed with materials having a higher resistance to current attack, such as rock or concrete blocks. From the calculations it was concluded that, from a hydraulic point of view, it does not make much difference whether this further narrowing is done prior to, simultaneously with, or after the construction of the sill in the deeper parts of the eastern channel.

C.11 Final cross-dam profile

C.11.1 Design criteria

In Chapter C.1 functional and structural requirements (short- and long-term) have been described. These requirements are worked out in more detail hereafter, insofar as they have not been discussed in the previous sections.

Overtopping versus non-overtopping

Tidal currents across the dam should be avoided, except during very high astronomical high water-levels. Occasional overtopping would not be detrimental to the accretion process. A dam which would create a "complete" hydraulic separation between the Sandwip and Hatia Channels would have to be very high (PD + 10-11 m) and would be subjected to heavy wave attack; a lower dam would be subjected to less wave attack. A higher non-overtoppable dam has been found more expensive than an overtoppable dam with (slightly) lighter protection. As the lower overtoppable dam fulfils the functional requirements, this is preferable to a non-overtoppable higher dam.

As an acceptable design level, an astronomical high water-level which is exceeded for 1 % of the high waters has been selected. This implies that overtopping of the final dam would occur only around seven times per year during a short time (not more than 1 to 1½ hours). Occasional wind set up (which can be substantial in case of severe cyclones) could double the number of occasions. For the various dam sections in the channels to be crossed the design crest level has been determined as indicated in Table C.9. In this table the maximum expected overflow height (in case of severe cyclones) and the expected difference in water-levels under those circumstances have also been indicated. It can be seen that the head difference is larger for the western dam (2 m) than for the eastern dam (1 m), which is caused by different wind setup on each side of the respective dam sections.

The dam section on Char Pir Baksh will be connected to the future polder dike system. The crest level has therefore been fixed at PD + 8.5 m (see Annex F, Section F.4.3).

As overflow of the dam takes place only when water-levels on both sides of the dam are very high, the overflowing water sheet has no chance to attack the already accreted land. The same applies to waves. The dam and its protection should however be designed to withstand the current and wave attacks to be expected.

Permeability of the dam

In order not to disturb the accretion process of the closure of the various channels, seepage currents through the dam should be limited. The exit velocity of seepage flow through the dam should not exceed 0.25 m/s. This criterium is satisfied by an earth dam (as described in Chapter 10) on the shoals. For a rockfill dam, especially one with larger units, additional measures are required to restrict the seepage flow through the dam. This could be achieved by filling the voids within the stone/block structure with finer materials, or by a surrounding or adjoining (overtoppable) earth structure. During the final design stage further attention will be paid to this problem, in order to determine the most appropriate solution.

Table C.9 - Design crest levels of dam sections

Chainage (km)	Dam section	Design crest level	Assumed Maximum overflow height	Water-level difference
0-5.3	Western closure dam (shoal and channel)	PD + 6 m	3 m	2 m
	Central channels	PD + 8.5 m		
14-22	Eastern closure dam (shoal and channel)	PD + 5 m	2 m	1 m

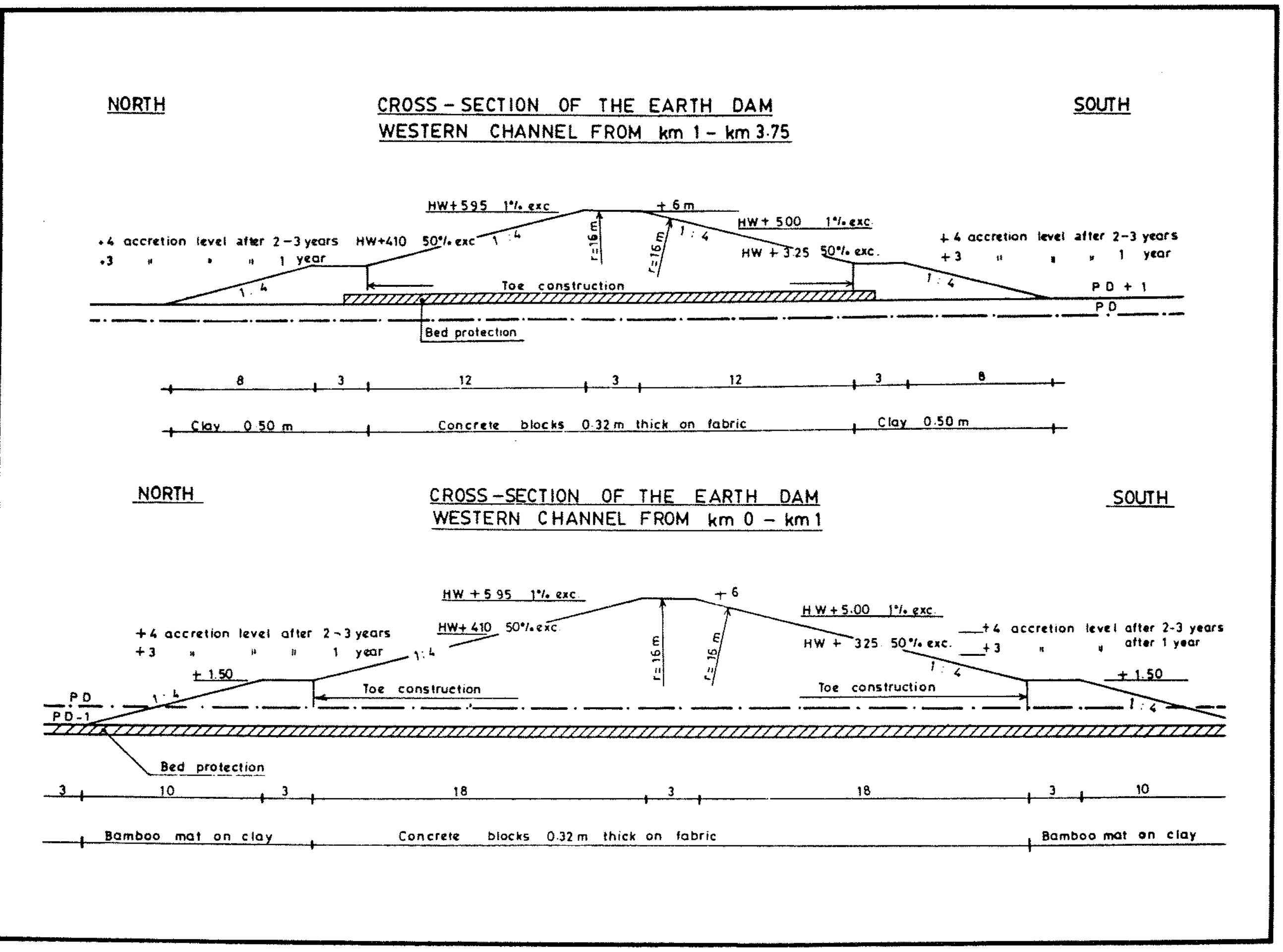
C.11.2 Geometry and structural design of dams on the shoals

The geometry of the earth dam on the shoals is primarily determined by hydraulic and geotechnical considerations. In order to guide the overflow due to water-level differences and wave run-over, relatively smooth slopes (1:4) and a rounded crest (with a radius of more than 15 m) are required. As a result, the drag and lift forces caused by a combination of tidal currents and waves remain within acceptable limits. A slope of 1:4 also satisfies the soil mechanics stability criteria (not steeper than 1:3.5).

The dams on the shoals can be built with silty sand from Char Lakhi (for the western dam) and the eastern shoal (cum island) near Char Pir Baksh. This material is not resistant against water and wave attack, and has therefore to be protected. In view of the expected accretion, the lower parts (below PD + 1.5 m to PD + 3.0 m, depending on the level of the foreland) require only temporary protection. Clay and bamboo matting are considered adequate in this zone. Above PD + 1.5 m a more durable revetment has to be provided.

This revetment could consist of concrete blocks (dimensions 0.50 x 0.50 x 0.32 m) to be placed on a filter fabric. Heavier blocks are required at the heads of the earth dams. Alternative solutions with for instance sand asphalt or gabions filled with bricks are also possible. The cost of these alternatives is somewhat higher than the cost of concrete blocks. Therefore at this stage concrete blocks have been selected for the upper part of the revetments (including the crests) of the earth dams.

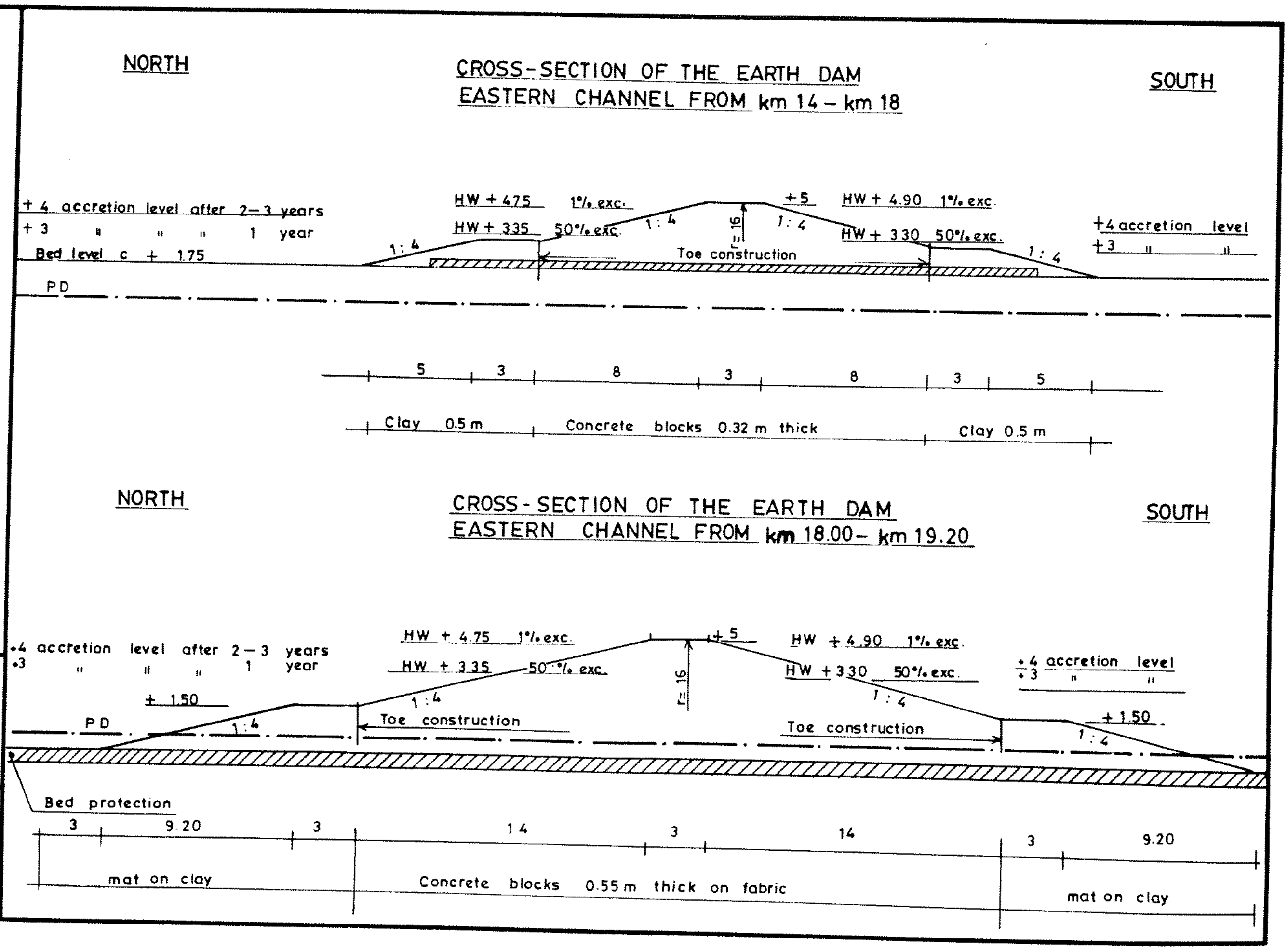
Cross-sections over the earth dams on the shoals in the western and eastern channels are given in Figures C.40 and C.41.



SANDWIP CROSS-DAM FEASIBILITY STUDY

CROSS-SECTIONS OF EARTH DAM (WESTERN CHANNEL)

FIG. C.40



SANDWIP CROSS-DAM FEASIBILITY STUDY

CROSS-SECTIONS OF EARTH DAM (EASTERN CHANNEL)

FIG. C.41

C.11.3 Geometry and structural design of dam in the channels

The rockfill dams with underlying sill by themselves almost satisfy the overtopping criterium as far as the crest levels are concerned. For the western channel the height of the stone dam, required for closure purposes, is 1.5 m lower than the design final crest level (Table C.9). A relatively small increase in the volume of the rockfill dam would transform the closure dam into a final dam as far as the volume of rock is concerned. For the eastern dam this difference is only 0.5 m, so the increase in volume is almost negligible.

It would be possible to surround the rockfill dam sections with earth, or to construct an adjoining earth dam, so that the final dam section in the closure-gaps would more or less match the dam section on the shoals. For the western closure, a combination of a low rockfill closure dam with an adjoining higher earthfill dam with revetment would cost about as much as a heightened rockfill dam, but the construction time would be substantially longer and the risks of damage during construction correspondingly higher. (More considerations on risks are given in Section C.12). Therefore preference has been given to an increase of the dimensions (width and height) of the rockfill closure dams to allow them to function as final dam sections as well.

The rockfill gradings required to effect the closures are too small to withstand (extreme) current forces and wave attack in the final stage. The sizes of the blocks of the outer protective layers therefore have to be increased. The following load combinations are to be expected:

(a) wave forces at water-levels below the crest level;

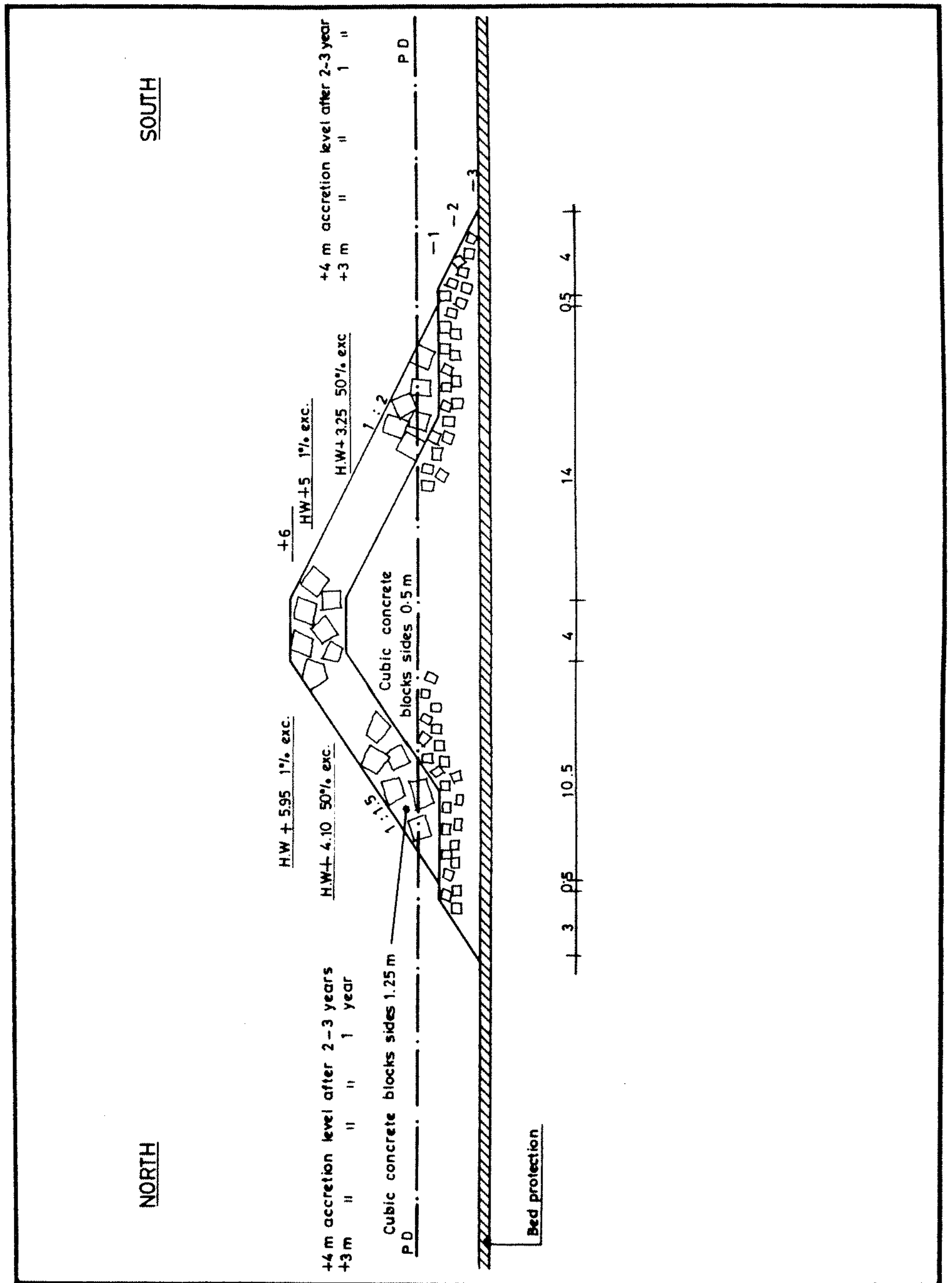
(b) current and wave forces at water-levels above the crest level.

For the western dam the latter combination requires the heaviest blocks, while for the eastern dam the wave forces alone are decisive. The block sizes (assuming concrete blocks) have been determined at 1.25 m (weight 4.3 t) for the western dam, and 1.00 m (weight 2.2 t) for the eastern dam.

Owing to substantial head differences for the western dam, exit gradients of seepage water through the dam may lead to instability of the slope, if the slope is taken at 1:1.5. To avoid this potentially dangerous situation the southern slope of the western dam section has been fixed at 1:2 for the time being. Cross-sections over the final dam sections are given in Figures C.42 and C.43.

The dam section on Char Pir Baksh will be connected to the future polder dike system. The crest level has therefore been fixed at PD + 8.7 m (see Annex F, Section F.4.3).

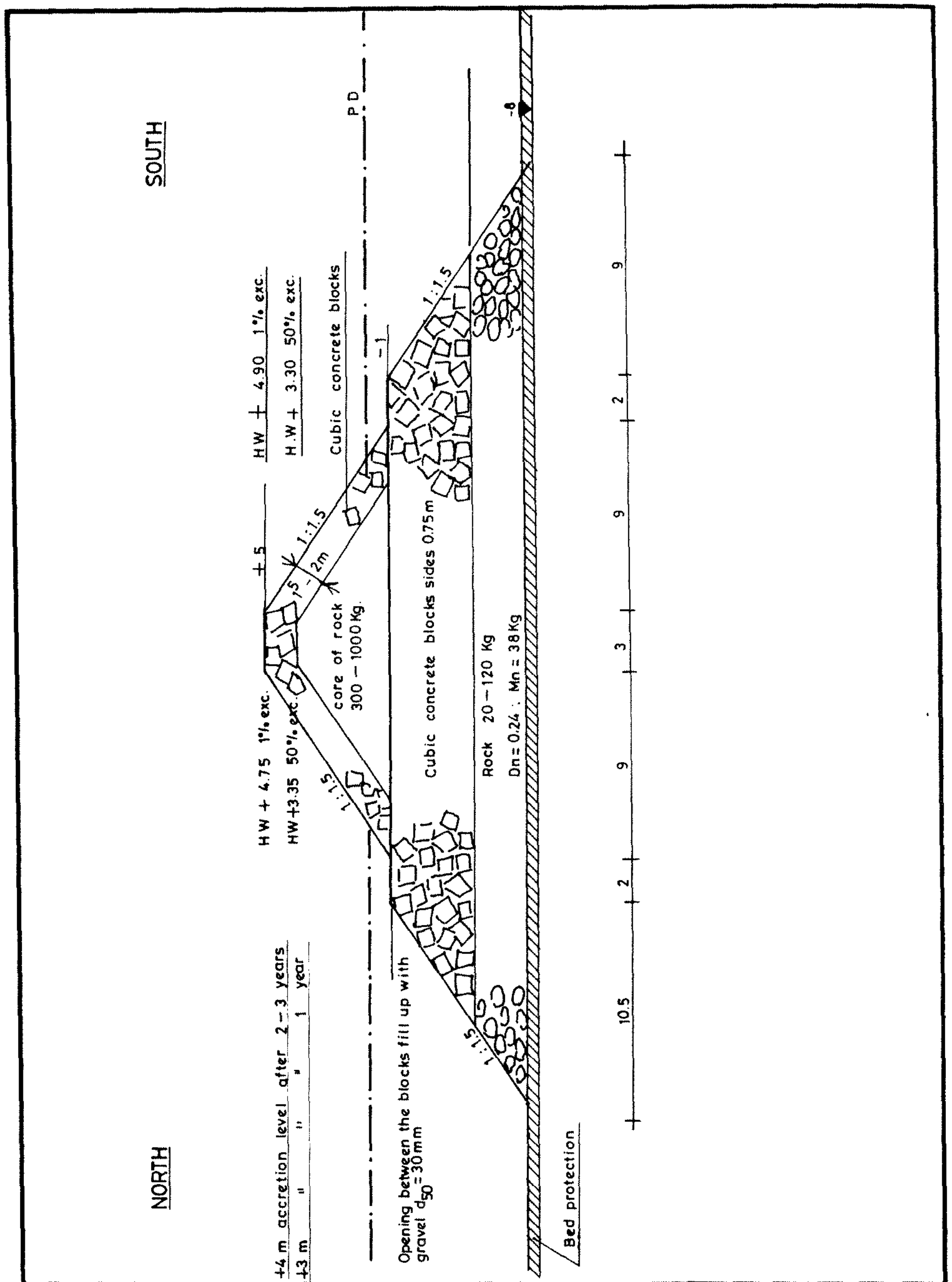
In the final design stage, when more data on waves should be available, the dimensions of the blocks in the outer layer of the dams should be checked again to see if they can be reduced. This will not lead to a reduction in the total volume of the blocks/rock, but smaller (and therefore lighter) blocks are easier to handle.



CROSS-SECTION OF THE CLOSURE DAM
WESTERN CHANNEL FROM km3.75-km5.25

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.42



CROSS SECTION OF THE CLOSURE DAM
EASTERN CHANNEL FROM km 19.20-km 22.00

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.43

Further savings may be achieved by reducing the crest width of the dam, and by increasing the steepness of the southern slope of the western dam from 1:2 to 1:1.5. Model investigations are required before these decisions can be made. A gunny bags closure (western channel) would not be stable by itself under wave attack, and would have to be protected with a rather strong revetment. It is estimated that a rubble stone revetment would have to be at least 1 m thick.

The caissons for a caisson closure (eastern channel) would have to be stable by themselves, also for post closure situations. Therefore they should be able to resist wave attack and differential water pressure in the final stage.

C.11.4 Dam on Char Pir Baksh

For the dam section on Char Pir Baksh the same slope as for the dam sections on the shoals has been adopted, which also satisfies the stability criterion. In view of the high level of Char Pir Baksh, water and wave attack will be considerably less than for the dam sections on the shoals. Protection of the sandy silt core with a layer of clay is considered sufficient.

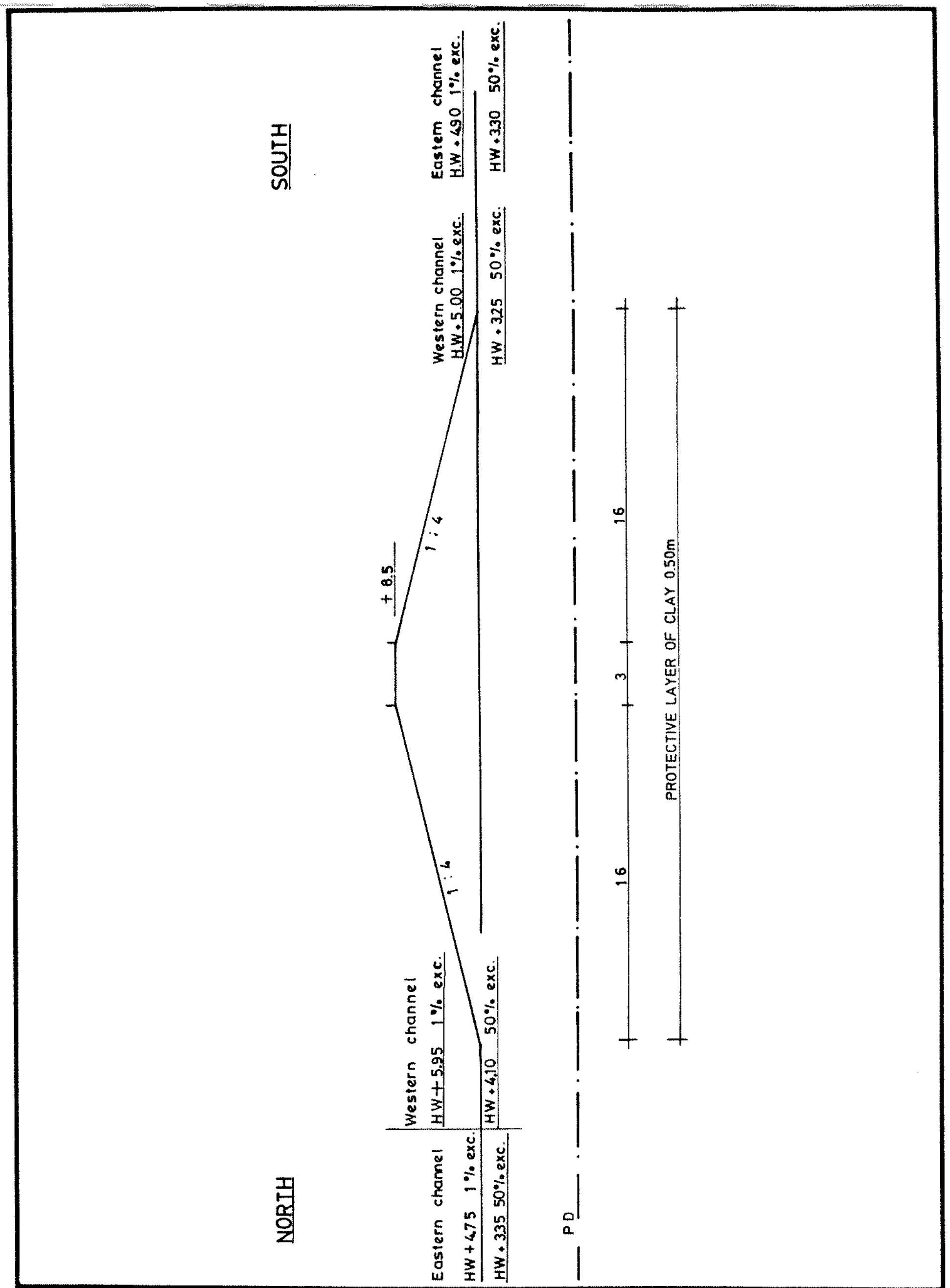
In view of the nature of the central closures, the same final dam section as on Char Pir Baksh will be applied here. Cross-sections over the dam section on Char Pir Baksh are given in Figure C.44.

C.12 Risk analysis and work methods

Final acceptance (or rejection) of the closure methods discussed in previous chapters should not be made before insight has been obtained in the risks related to those methods. Risks can result from the forces of nature (cyclones, waves, rainfall, etc.) or from human factors such as the selection of equipment or deviations from the specifications for materials.

Risks due to natural forces can be reduced to a certain extent by scheduling critical operations in seasons with a low probability of occurrence of hazardous natural events, such as cyclones, high water-levels, etc. Risks resulting from human factors can be reduced by selecting the proper materials, equipment, labour-force organization etc. Control of the human factors is basically the responsibility of the contractor(s).

Risks resulting from natural or human factors are not the same for the various closure methods discussed in previous chapters. A systematic approach has been adopted to trace and define the risks inherent with those closure methods. The final selection of the closure method should take the results of the risk analysis into account. In addition, the risk analysis also aims at identifying aspects which should receive special attention during the final design stage.



CROSS-SECTION OF THE EARTH DAM ON CHAR PIR BAKSH FROM km 5.25 - km 14.00

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.44

In this chapter the following aspects will be worked out:

- description of method of risk analysis;
- system description of closure constructions (including work methods);
- limit states and fault trees;
- analysis of risks and remedies;
- conclusion.

C.12.1 Description of risk analysis methods

Limit states (possible failure modes or limiting conditions) can be distinguished for various construction stages and the final stage of the closure construction(s); they form the basis of the risk analysis. Failure of (a part of) the construction may be caused, for instance, by:

- deviations in the size of materials applied;
- higher loads than expected;
- friction between different materials less than expected;
- late delivery of materials or failure of equipment;
- late adjustment of construction programme to changing natural circumstances.

For a proper understanding of the limit states it is necessary to have an idea of the construction methods of the various closure methods and parts thereof. A description of the work methods has therefore been included in the 'system description of closure constructions' (Subsection C.12.2). Subsequently the limit states will be defined. For every limit state the causes of failure will be determined and arranged in 'fault trees'. An estimate of the probability of failure of the entire structure can be made by analysing all possible causes of failure. Suggestions for (quick) remedial measures to overcome potential limit states will be made where appropriate.

C.12.2 System description of closure constructions (including work methods)

In previous chapters the following closure methods were discussed:

Western and eastern closure sections on shoals. Earthfill dam (incorporating a gunny bag section for construction purposes), with protective revetments of concrete blocks, sand asphalt or gabions.

Western closure section in gully

(a1) Closure dam of rock and/or concrete blocks; cross-section enlarged to allow the closure section to function as final dam section as well.

(a2) Closure dam of rock and/or concrete blocks; cross-section of closure dam is sufficient for closure purposes, but not for final situation - surrounding or adjacent soil structure with protective revetments is necessary in completed stage.

(b) Sill of rock and/or concrete blocks; final closure with gunny bags, subsequently to be surrounded with earth and protective revetment.

Eastern closure section in gully

(a) Closure dam of rock and/or concrete blocks; cross-section (slightly) enlarged to allow the closure section to function as final dam section well.
(b) Sill of rock and/or concrete blocks; final closure with caissons.

Central channels. Closures with gunny bags; final earthfill structure surrounding the closure profile, with a protective layer of clay. The dam section on Char Pir Baksh will consist of earth with a protective layer of clay.

The work methods envisaged for the various parts of the closure constructions are outlined below:

Bed protection on shoals: A bed protection mattress consists of filtercloth (composite woven and non-woven fabrics). Bamboos and reed rolls are fixed to the fabric to form a grid of compartments. Ballast materials (bricks, boulders or similar materials) are dumped in the compartments and subsequently covered with wire netting to prevent the ballast from being taken away by the currents. The mattress could be fabricated 'in situ' during low tides. This requires that all materials (fabric, bamboos, ballast, wire netting) be at hand when the shoals run dry. This can be achieved by transporting the materials to the work sites on flat barges during high water; as the water level drops the barges will become grounded, after which they can be unloaded manually. The fabrication of mattresses is also done manually. Joints (or rather overlaps) between mattresses can be made in a well-controlled manner.

Bed protection in gullies: Mattresses will have to be (partly) pre-fabricated on the shore. The bamboos and reed rolls will initially provide buoyancy and stiffness to the filter fabrics when they are floated from the construction yard to the gully locations to be protected. Once the mattress has been manoeuvred into position (around slack water), it will be sunk by dumping ballast onto the fabric from barges. Ballasting could be done manually or mechanically, depending on the size of the ballast, which is not the same for all locations. Mattress overlaps require special attention, and should be laid down by accurate surveying and/or by fixing floats at the corners of earlier placed mattresses. Accurate placing and verification of overlaps in the gullies is inherently more difficult than on the shoals and would require regular surveys and, possibly, post-construction maintenance.

Construction of sill: For the purpose of this description the sill is assumed to encompass all materials required to raise the level of the gullies such that the final closure construction can be built on top of this sill. The top levels of the sill are not the same for the various closure methods. The different levels are:

PD - 1 m for stone/rock closures (W & E).
PD + 0 m for gunny bag closure (W).
PD + 0.5 m for caisson closure (E).

Gradings of rock/concrete blocks vary for the western and eastern sills are indicated in the drawings included in Section C.8. A level of PD - 1 m can conveniently be reached with water-borne equipment, while levels above PD - 1 m probably have to be built by land-based equipment

(at least partly). The sill for a caisson closure should have a relatively smooth surface, which can be achieved during low water using manual labour.
Sills for entire stone/concrete block dams and the caisson closure require additional waterproofing measures after closure. This can be achieved by filling the voids with coarse gravel (or similar material) after completion of the closure.

Construction of dam of rock/concrete cubes: The (larger) rocks and/or concrete blocks can be dumped by trucks. A practical layer thickness is 2 m (which thickness has been used in the design calculations). To enable driving over the partly completed dam, the voids in the upper part of each layer have to be filled with crushed bricks. Protruding corners may be cut off manually. In view of the sizes of the concrete cubes, it may be attractive to carry out part of the dam construction with barge-mounted cranes and flat barges, which implies transporting the cubes over water to their location in the dam. The works have to be planned such that the last section of each layer can be completed during a period of neap tides, so that the most severe currents through the closure-gaps are avoided.

Construction of gunny bag closure dam: Gunny bags will have to be brought in and stockpiled on top of the (locally widened) sill. This should ideally be done during low water (when the sill will be dry or almost dry), implying that the bags have to be transported on barges from the shore to the stockpiles during high water. Adequate protection has to be provided to prevent erosion of the stockpiles by the currents through the open gap. Loading/unloading of barges and building up of stockpiles could be done manually. During low water at a suitable neap tide, all stockpiles have to be 'turned 90 degrees' by labourers. The estimated number of labourers for this operation is 12 000 to 15 000. In view of the a smaller labour requirements for preceding work, this will pose considerable management problems.

Placing of caissons: Caissons would have to be precast in a (temporary) dock on Sandwip, well ahead of the closure period. Caissons should be placed on the sill around high slack water. Sufficient time should be allowed for manoeuvring and comprehensive anchoring facilities would be required.
Caissons can be sunk by opening valves or gates in the caissons. Immediately after sinking, sufficient fill material should be placed inside the caisson so it can resist the horizontal forces and overturning moments resulting (mainly) from differential water-levels. These will occur near high slack water, particularly towards the final closure stage. Filling should consequently be done very fast, which will restrict the number of caissons to be placed in a single operation.

Construction of dam profile on shoals: The dams consists of earth, which can be excavated locally (Char Lakhi and the eastern shoal near Char Pir Baksh). Mechanical excavation and transport is probably the only answer to the requirement to make fast progress. During construction of the earth structures a protection bund of gunny bags will be required, which can be made with earth manually excavated on the shoals. Effective dumping of earth at the advancing dam end will have to be restricted to low water periods; to resist erosion during high water periods temporary protection of the dam end may be required. As soon as possible the earth body should be protected with a revetment. Most of this work could be executed by manual labour.

Other construction stages, such as closures of the central channels and the dam on Char Pir Baksh, are not discussed here, but will have to be scrutinised when more details on depth contours etc. are known.

C.12.3 Limit states and fault trees

A limit state is defined as a possible failure mode of (or part of) the closure structure. A limit state may lead to total failure of the structure. Relevant limit states are (Figure C.45):

- erosion of crest;
- erosion of slope;
- loss of micro stability;
- loss of macro stability;
- internal erosion;
- settlement of entire dam body;
- formation of scourhole;
- liquefaction of subsoil;
- failure of bed protection.

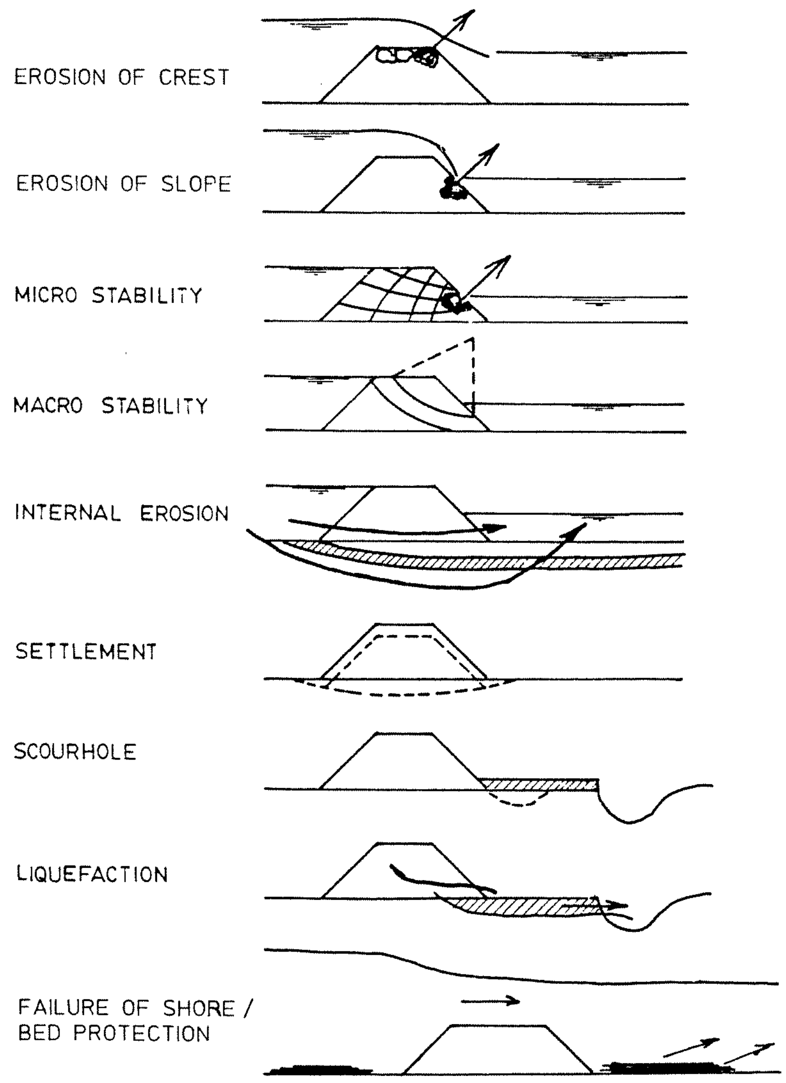
Failure or, in other words, exceedance of a limit state, can be visualized by means of a fault tree. In a fault tree the relationship between subsystem failures, component failures and initiating events can be presented. The probability of occurrence of initiating events can generally be obtained from two sources: historical observations and calculations. For the creation of a fault tree it is necessary to investigate all possible failure modes and their mutual relationships.

The main fault tree contains the various limit states (Figure C.46). For each separate limit state a more detailed fault tree has been constructed.

C.12.4 Analysis

The detailed fault trees for every limit state have been analysed. Owing to lack of relevant statistical data no probabilistic calculations have been made. For this reason the probability of occurrence of a certain failure mechanism has not been given as a percentage, but with the statements 'large' and 'small'. ('Large' corresponds with a high probability of failure, 'small' with a low probability.) Experience with similar works formed the basis for assigning the attributes 'large' and 'small' to certain probabilities. Before summarising the results of the analysis some general remarks should be made.

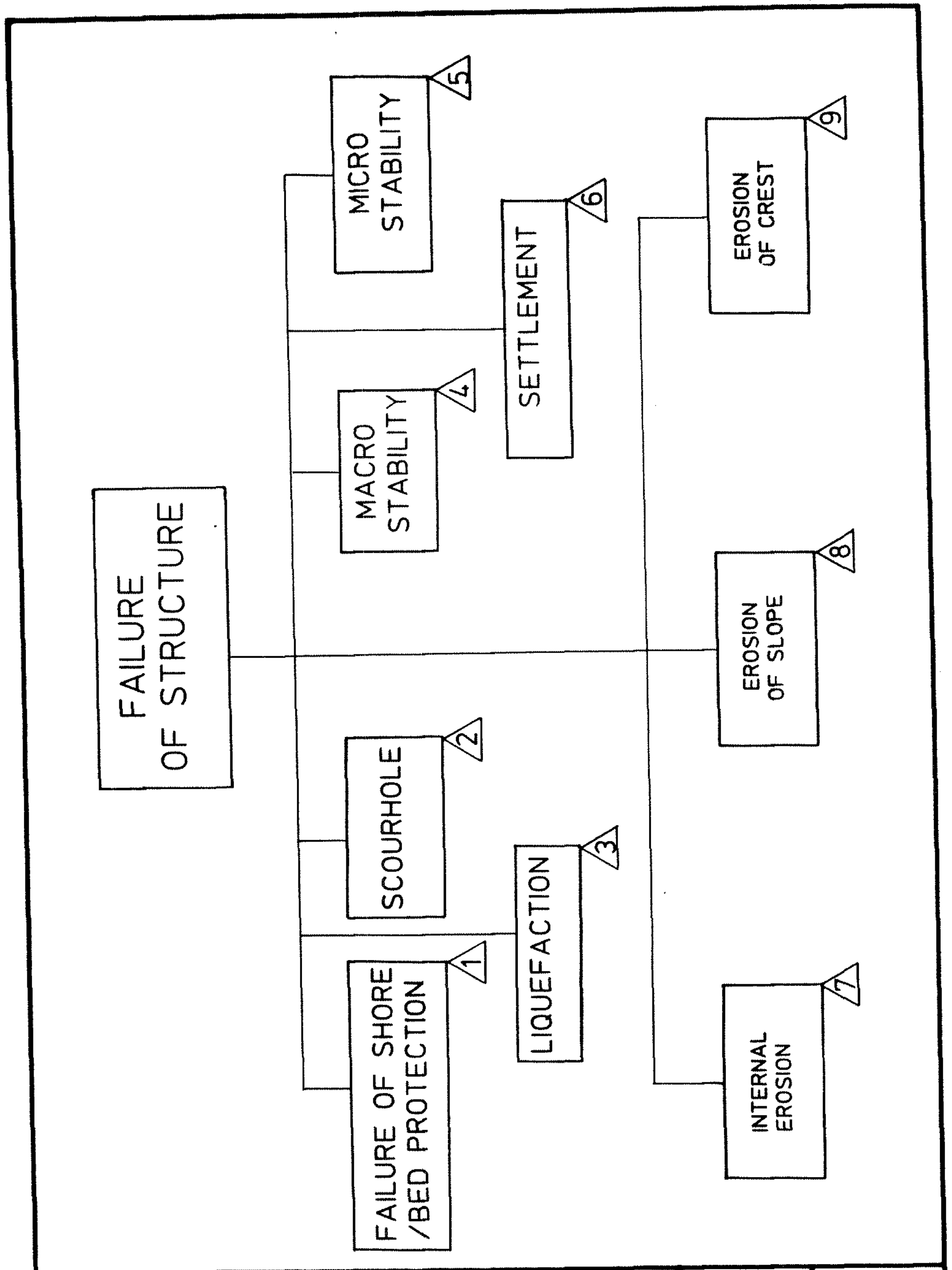
- The designs of various alternatives have been based on the acceptance of damage under certain circumstances. Some damage is thought to be acceptable in view of:
  - . the temporary function of the closure-dams (after land accretion there will be no or far less current and wave attack);
  - . the fact that minor damage could easily be repaired, especially during the construction period.



LIMIT STATES

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.45



MAIN FAULT TREE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.46

The extent of damage could however not yet be determined due to lack of data and unavailability of model investigation results.

- No explicit safety factors have been introduced concerning the hydraulic aspects of the various alternatives. The consequence of this approach is different for each alternative. In the final design stage the choice of the safety factor should be based on an analysis of the damage to be allowed. An economic optimum may be found.

- The design of the bed protection is based on the assumption that liquefaction of the subsoil will not occur. The results from the geotechnical research point in this direction, but the amount of relevant data is very small. Unforeseen occurrence of liquefaction could have disastrous consequences.

- Damage to the bed protection caused by anchors, cables etc. is a serious problem. Such damage may lead to failure of the bed protection (or worse) in the critical stage of the closure. Adequate inspection and repair procedures are necessary.

- The time schedule of the various alternatives should be based on the requirement to minimise the exposure time in a critical construction stage. If possible the critical stage should be "passed" in a period of neap tides.

The results of the analysis are presented in Tables C.10a-k. Each table contains the following:

- name of the limit state;
- main cause(s) of failure;
- consequence of failure (what is the resulting problem);
- probability of occurrence;
- possible solutions.

<u>Scheme 1 - (Table C.10a and b):</u>
<u>Sill and closure dam of concrete blocks with surrounding or adjacent earth structure (western channel)</u>

- Because of the critical stage between PD-2 m and PD+0 m, loss of hydraulic stability of the bed protection is likely to occur. Enlargement of the size and/or density of the stones of the bed protection is to be considered in the final design stage. Construction time of this critical stage should be as short as possible, and neap tides should be fully utilised.
- The same applies to the stability of the concrete blocks between PD-2 m and PD+0 m.
- Settlement of the final dam structure (if surrounded with earth) has to be expected owing to voids remaining between the stones/concrete cubes. This can be prevented by placing the earth structure adjacent to the closure structure.

Scheme 1 is not recommended in view of the large risks involved.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Failure shore/ bed protection	size/density stones	- erosion ballast material	large	- heavier ballast material - acceptance damage, repair - organisation
	current velocity etc.	- current velocity higher than expected: • erosion ballast material • loss of stability edges	large	- heavier ballast material - ballast on edges - heavy edges - organisation
	no overlap between mattresses	- loss of bedmaterial - loss of stability edges	small	- procedure inspection overlap - soundings - material and equipment for spare mattress standby
	damage (anchors)	- loss of bedmaterial	large	- see no overlap between mattresses
	liquefaction	- see limit state liquefaction	small	- see limit state liquefaction
	external factors	- delivery material too late	small	- organisation logistics - size stock piles
Liquefaction	material characteristics (sandlayers)	- failure shore/bed protection - erosion bed material	small	- reliable geotechnical research - extension shore/bed protection
Macro stability	material characteristics (φ)	- safety factor low	small	- slope less steep - acceptance small damage
	liquefaction	- deep slip circle	small	- see limit state liquefaction
Settlement final profile	% voids between concrete blocks and coarse gravel	- penetration soil in voids in final stage	large	- procedure inspection during fill with coarse gravel plus procedure inspection during construction final profile - elevation crest level

Table C.10a Main risks and possible solutions
West, deep part: Sill and closure dam of concrete blocks with soil.

C.116

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Erosion slope/ crest concrete block dam	current velocity etc. (reliability data, three dimensional flow, progress of work)	- safety factor low	large	- heavier elements - acceptance damage, repair - organisation
	accuracy construction	- exposed stones at PD-1, erosion, during construction	small	- see current velocity etc. - procedure egalisation
		thickness layers dumped with trucks larger than expected	small	- see current velocity etc. - procedure control thickness
	external factors	- delivery material too late, erosion during construction	small	- organisation logistics - size stock piles - see current velocity etc.
Erosion slope/ crest final profile	current velocity etc.	- safety factor low	small	- heavier elements - acceptance damage, repair - heavy fabric under blocks
	accuracy construction	- erosion exposed blocks	small	- see current velocity etc.
	settlement	- loss of pattern blocks, erosion blocks	large	- see current velocity etc.

Table C.10b Main risks and possible solutions
West, deep part: Sill and closure dam of concrete blocks with soil, protected by blocks.

C.117

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Failure shore/ bed protection	size/density stones	- erosion ballast material	small	- acceptance damage, repair - heavier ballast material - organisation
	current velocity etc. (reliability data, three dimensional flow progress of work)	- current velocity higher than expected: • erosion ballast material • loss of stability edges	small	- heavier ballast material - ballast edges - heavy edges - organisation
	no overlap between mattresses	- loss of bedmaterial loss of stability edges	small	- procedure inspection overlap - soundings - material and equipment for spare mattress standby
	damage (anchors)	- loss of bedmaterial	large	- see no overlap between mattresses
	liquefaction	- see limit state liquefaction	small	- see limit state liquefaction
	external factors	- delivery material too late	small	- organisation logistics - size stock piles
Liquefaction	material characteristics (sandlayers)	- failure shore/bed protection - erosion bed material	small	- reliable geotechnical research - extension shore/bed protection
macro stability	material characteristics (φ)	- safety factor low	small	- slope less steep - acceptance small damage
	liquefaction	- deep slip circle	small	- see limit state liquefaction

Table C.10c Main risks and possible solutions
West, deep part: Concrete blocks.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Erosion slope/ crest concrete block dam	current velocity etc. (reliability data, three dimensional flow, progress of work)	- safety factor low, construction stage final stage	large small	- heavier elements - acceptance damage, repair
	accuracy construction	- exposed stones at PD-1, erosion during construction	small	- see current velocity etc. - procedure egalisation
		thickness layers dumped with trucks larger than expected	small	- see current velocity etc. - procedure control thickness
	external factors	- delivery material too late, erosion during construction	small	- organisation logistics - size stock piles - see current velocity etc.

Table C.10d Main risks and possible solutions
West, deep part: Concrete blocks.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Failure shore/ bed protection	size/density stones	- erosion ballast material	small	- heavier ballast material - acceptance damage, repair
	current velocity etc. (reliability data, 3-dimensional flow, progress of work)	- current velocity higher than expected: • erosion ballast material • loss of stability edges	small	- heavier ballast material - ballast on edges - heavy edges - organisation
	no overlap between mattresses	- loss of bedmaterial loss of stability edges	small	- procedure inspection overlap - soundings - material and equipment for spare mattress standby
	damage (anchors)	- loss of bedmaterial	large	- see no overlap between mattresses
	liquefaction	- see limit state liquefaction	small	- see limit state liquefaction
	external factors	- delivery material too late	small	- organisation logistics - size stock piles
liquefaction	material characteristics (sandlayers)	- failure shore/bed protection erosion bed material	small	- reliable geotechnical research - extension shore/bed protection
macro stability	material characteristics (ψ)	- safety factor low	small	- slope less steep - acceptance small damage
	friction between sandasphalt and gunny bags	- loss of gunny bags	small	- increase roughness sand asphalt
	liquefaction	- deep slip circle	small	- see limit state liquefaction

Table C.10e Main risks and possible solutions
West, deep part: Sill of concrete blocks with gunny bags.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Erosion slope/ crest concrete block dam	current velocity etc. (reliability data)	- safety factor low	large	- heavier elements - acceptance damage, repair
	accuracy construction	- exposed stones at PD, erosion during construction	small	- see current velocity etc. - procedure egalisation
	external factors	- delivery material too late, erosion during construction	small	- organisation - size stock piles - see current velocity etc.
		failure final closure	large	- organisation - see current velocity etc.
Erosion sand-asphalt	external factors	- failure final closure	large	- organisation - see current velocity etc.
Erosion of gunny bags in stock piles	three-dimensional flow	- loss of gunny bags loss of weight gunny bags desintegration gunny bags	large	- extra gunny bags in in stockpiles - protection toe stock piles
	external factors	- failure final closure	large	- organisation - acceptance damage, repair
Erosion gunny bags in dam-profile	external factors	- failure finishing damprofile	large	- organisation - acceptance failure, retry
Erosion final dam profile	current velocity etc. (reliability data)	- safety factor low	small	- heavier elements - acceptance damage, repair
	accuracy construction	- erosion exposed stocks	small	- see current velocity etc.
	external factors	- delivery material too late	small	- organisation - size stockpiles - acceptance damage, repair
		production too low	small	- organisation - spare parts equipment - acceptance damage, repair

Table C.10f Main risks and possible solutions
West, deep part: Sill of concrete blocks with gunny bags.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Failure shore/ bed protection	size/density stones	- erosion ballast material	small	- heavier ballast material - acceptance damage, repair
	current velocity etc. (reliability data, 3-dimensional flow, progress of work)	- current velocity higher than expected: • erosion ballast material • loss of stability edges	small	- heavier ballast material - ballast on edges - heavy edges - organisation
	no overlap between mattresses	- loss of bedmaterial loss of stability edges	small	- procedure inspection overlap - soundings - material and equipment for spare mattress standby
	damage (anchors)	- loss of bedmaterial	large	- see no overlap between mattresses
	liquefaction	- see limit state liquefaction	small	- see limit state liquefaction
	external factors	- delivery material too late	small	- organisation logistics - size stock piles
Liquefaction	material characteristics (sandlayers)	- failure shore/bed protection erosion bed material	small	- reliable geotechnical research - extension shore/bed protection
macro stability	material characteristics (φ)	- safety factor low	small	- slope less steep - acceptance small damage
	liquefaction	- deep slip circle	small	- see limit state liquefaction

Table C.10g Main risks and possible solutions
East, deep part: Conrete blocks.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Erosion slope/ crest concrete block dam	current velocity etc. (reliability data, three dimensional flow, progress of work)	- safety factor low, construction stage final stage	small small	- heavier elements - acceptance damage, repair
	accuracy construction	- exposed stones at PD-1, erosion during construction	small	- see current velocity etc. - procedure egalisation
	external factors	- thickness layers dumped with trucks during construction	small	- see current velocity etc. - procedure control thickness
		- delivery material too late, erosion during construction	small	- organisation logistics - size stock piles - see current velocity etc.

Table C.10h Main risks and possible solutions
East, deep part: Concrete blocks.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Failure shore/ bed protection	size/density stones	- erosion ballast material	small	- heavier ballast material - acceptance damage, repair
	current velocity etc. (reliability data, 3-dimensional flow, progress of work)	- current velocity higher than expected: • erosion ballast material • loss of stability edges	small	- heavier ballast material - ballast edges - heavy edges - organisation
	no overlap between mattresses	- loss of bedmaterial loss of stability edges	small	- procedure inspection overlap - soundings - material and equipment for spare mattress standby
	damage (anchors)	- loss of bedmaterial	large	- see no overlap between mattresses
	liquefaction	- see limit state liquefaction	small	- see limit state liquefaction
	external factors	- delivery material too late	small	- organisation logistics - size stock piles
		final closure fails	large	- heavier ballast material - acceptance damage repair
liquefaction	material characteristics (sandlayers)	- failure shore/bed protection erosion bed material	small	- reliable geotechnical research - extension shore/bed protection
macro stability	accuracy sill	- flatness sill insufficient	large	- procudure egalisation

Table C.10i Main risks and possible solutions
East, deep part: Sill of concrete blocks plus caissons.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Macro stability	weight, size, caisson	- safety factor low - construction stage - final stage	large large	- fill caisson as soon as possible - increasing weight
	liquefaction	- deep slip circle	small	- see limit state liquefaction
	current velocity etc. (reliability data, three dim effects)	- see weight, size caisson	small	- see weight, size caissons
Settlement	wash-out	- loss of stability loss of impermeability	small	- procedure inspection - procedure removal sand
Erosion slope/ crest concrete block dam	current velocity etc. (reliability data, three dimensional effects, progress of work	- safety factor low	small	- heavier elements - acceptance damage, repair
	accuracy construction	- exposed blocks at PD	small	- heavier elements - procedure egalisation
	external factors	- failure final closure	small	- see current velocity etc.

Table C.10j Main risks and possible solutions
East, deep part: Sill of concrete blocks plus caissons.

LIMIT STATE	CAUSE	PROBLEM	PROBABILITY OF OCCURANCE	SOLUTION
Failure shore/ bed protection	liquefaction in deep part of gully	- failure shore/bed protection - erosion bed material	small	- reliable technical research - extension shore/bed protection
Macro stability	liquefaction in deep part of gully	- deep slip circle	small	- see limit state liquefaction
Erosion slope/ crest gunny bags, construction stage	current velocity etc. (reliability data, three dimensional flow, progress of work)	- loss of gunny bags, especially at front - loss of soil	small small	- acceptance damage, repair - acceptance damage, repair
Erosion slope/ crest final profile, - concrete blocks	current velocity etc. (reliability data)	- safety factor low	large	- heavier elements - acceptance damage, repair
	accuracy construction	- erosion exposed blocks	small	- see current velocity etc. - heavy fabric under blocks
	settlement	- loss of pattern blocks, erosion blocks	small	- see current velocity etc.
Erosion slope/ crest final profile - sandasphalt	current velocity etc. (reliability data)	- safety factor low	large	- acceptance damage, repair
Erosion slope/ crest final profile - gabions	accuracy construction	- loss of interaction between gabions loss of stiffness individual gabion, deformation and loss gabion	small	- procedure inspection

Table C.10k Main risks and possible solutions West and East, part on shoals.

Scheme 2 - (Table C.10c and d):
Sill and closure dam entirely made of stone/concrete blocks (western channel)

- Because of the critical stage between PD-2 m and PD+0 m, loss of hydraulic stability of the bed protection may occur. Enlargement of the size and/or density of the stones of the bed protection is to be considered in the final design stage. Construction time of the critical stage should be as short as possible, and neap tides should be fully utilised.
- The same applies to the stability of the concrete blocks between PD-2 m and PD+0 m.
- Application of a layer thickness of times the nominal block diameter on the crest should be considered in the final design stage.

Scheme 2 is judged to be feasible as far as the risks are concerned.

Scheme 3 - (Table C.10e and f):
Sill of concrete blocks, final closure with gunny bags (western channel)

- The effect of three-dimensional flow has been taken into account. In case of a successful final closure the probability of failure of the shore/bed protection is small, but damage may still occur, which will have to be repaired quickly.
- The time schedule makes it necessary to pass the critical stage of the sill during a period of (high) post-monsoon tides. This requires that larger concrete blocks have to be used than would otherwise be required (schemes 1 and 2).
- Protection of the toe of the stockpiles is necessary.
- The probability of failure of the final closure is large in view of the size of the closure-gap. Failure of the final closure will lead to (subsequent) serious damage of the bed protection and the sill.

Scheme 3 is not recommended in view of the large risks involved.

Scheme 4 - (Table C.10g and h):
Sill and closure dam entirely made of stone/concrete blocks (eastern channel)

- The critical stage occurs at a sill level of PD-3 m. For slightly lower and higher sill levels the prevailing head differences are considerably lower. For this reason the hydraulic risks are relatively small.
- In the final stage the safety factor is small. Application of a layer thickness of 2 times the nominal block diameter should be considered in the final design stage.

Scheme 4 is considered to be feasible as far as the risks are concerned.

Scheme 5 - (Table C.10i and j):
Sill of concrete blocks, caisson closure

- Failure of the final closure is considered to be likely in view of the time schedule required. Failure leads to serious damage to the sill and the bed protection.
- The stability of the caissons prior to filling is not sufficient. The time required to fill the caissons is relatively long, while it should be as short as possible.
- Damage to the bed protection will be large due to the use of tugs with cables and mooring pontoons with anchors.

Scheme 5 is not recommended in view of the large risks involved.

Scheme 6 - (Table C.10k):
Dam sections on shoals (west and eastern channels)

- No serious problems are expected.
- A protection of concrete blocks or gabions involves fewer risks than a protective layer of sand-asphalt.

The entire structure is considered to be feasible as far as the risks are concerned.

C.12.5 Recommendation

From the foregoing risk analysis it can be concluded that for both the western and eastern closures a stone closure involves fewer risks than other closure methods. The risks for a stone closure can be reduced (if desired) by applying heavier closure units, but this will involve extra costs. The other closure methods discussed in this and earlier chapters are not recommended.

C.13 Construction of the entire cross-dam - synthesis

C.13.1 Summary of the foregoing

The main conclusions reached in the previous sections can be summarized as follows.

The closure sequence to be preferred, on technical grounds, is the central closures first, then the western closure and lastly the eastern closure.

In the pre-selection of closure methods, the following methods were found to be feasible in principle:

- For the western closure: Feni-type closure or a stone closure;
- For the eastern closure: a stone closure or a caisson closure;
- For the central closure: closure methods applied earlier in Bangladesh.

The methods for the western and eastern closures require construction of a sill consisting of rock or concrete blocks prior to the actual closures. Furthermore, bed protection works will be required in the final closure gaps for various closure methods.

For the narrowing of the channels on the shoals, preference has been given to earthfill dikes, adequately protected against currents and waves.

Measures have been discussed to transform the (temporary) dike and closure dam sections into final dam sections.

Finally, from a risk analysis it is concluded that a stone closure is the safest solution, both for the western and the eastern channels.

C.13.2 Additional considerations for final selection

The picture which emerged from Sections C.4-C.12 is still fragmented, in the sense that it defines problems and formulates solutions for separate components of the Sandwip cross-dam. Giving solutions for separate components of the dam, however, does not imply that a combination of the several solutions will yield the best solution for the project as a whole. In making the final choice from the partial solutions, consideration should be given to two important aspects: the interrelationship between closures, and the flexibility to adapt to changing circumstances.

The various closures (central, western and eastern) should not be viewed as separate closures, for which a unique solution is the best answer for each closure. They should rather be viewed as a number of closures forming part of a single project. The choice of a particular closure method for one of the closures may involve the use of certain equipment, which can also be employed for the other closure(s). The same applies for the use of materials. For instance, the development cost for a quarry in the Chittagong Hill Tracts, if one can be found, may not be justified economically for one single closure. This picture may change however if two closures of the same type are concerned.

Flexibility of the dam design is essential, as the channel-shoal-island system is not a stable one. Gullies tend to shift continuously. Re-distribution of discharges over the various gullies in the future is also possible. In making the final decision on the closure-method(s) to be applied, it should be realised that adaptation of the closure works, either prior to or during construction, should be possible without drastic changes in the work methods or programmes.

C.13.3 Synthesis

Reviewing all the partial conclusions of previous chapters and taking into account the additional considerations mentioned above, the following scheme is judged to be the best suited for the implementation of the Sandwip cross-dam.

Sequence of closures. Technically speaking the western channel should be closed prior to the eastern channel. To arrest the present tendency of enlargement of the central channels, these have to be closed first. The preferred sequence then will be as indicated Subsection C.13.1.

Closure methods. Gradual vertical stone closures (with rock and/or concrete blocks) are preferred for both the western and eastern closures. In view of the large quantities of "hard" materials involved in the actual closure-dam construction and underlying sills, substantial cost can be expected for:

- development of a quarry;
- establishment of a concrete block plant;
- mobilisation of land-based and marine equipment;
- loading/unloading facilities.

Fast execution. The channel-shoal-island system is subject to changes, both prior to and after the start of construction works. In order to minimise changes during construction, and their consequences for the cost of the works, the whole closure-dam, including closure of the central channels, should be realised in as short a time as possible after the construction works have commenced. For this reason, and also to reduce the cost for separate closures, it is highly advisable to implement the entire cross-dam under one single contract with an experienced and financially strong contractor. An exception may be made for the dry earthworks on Char Pir Baksh, which may be contracted separately without affecting the progress of the closure works.

Crest levels. As occasional overtopping of the cross-dam can be allowed, except on Char Pir Baksh, the crest level of the dam sections in the eastern and western channels can be kept relatively low, implying considerable cost savings compared to a dam with a high, "non-overtoppable" crest. A longitudinal section over the proposed cross-dam is given in Figure C.47.

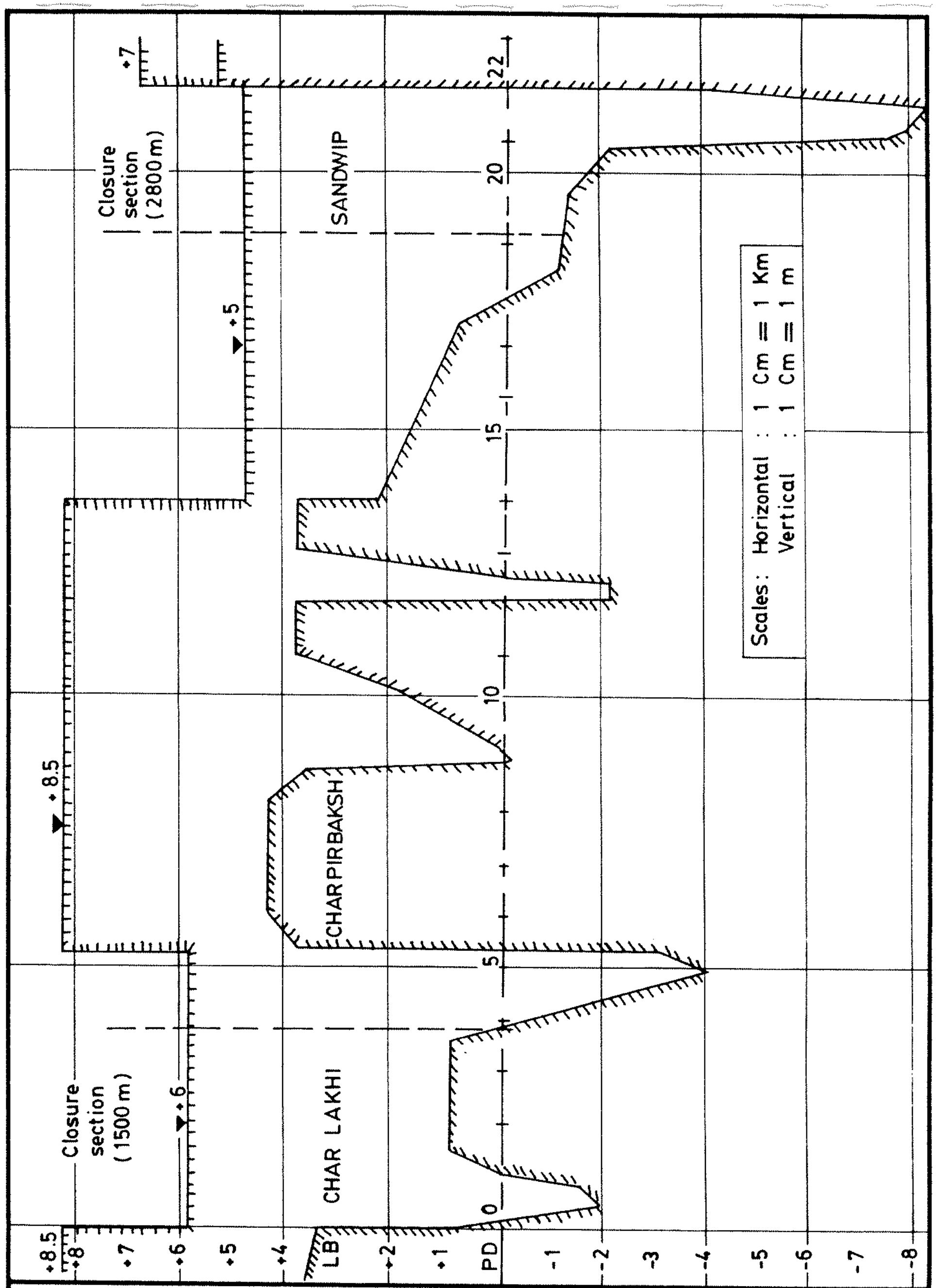
Final cross-sections. The volume of the closure-dam sections (west and east) is almost sufficient for a final dam section (in the form of a breakwater-like construction). A slight increase in the quantities of the closure-dams and proper dimensioning of the rocks (or concrete blocks) would transform the actual closure-dams into final dam sections. This option is attractive, both from the cost and from the scheduling point of view, and is therefore recommended. The dam sections on the shoals should consist of earthfill dikes with protective revetments. The dam section on Char Pir Baksh and the adjacent shoals could consist of earth with a protective layer of (turfed) clay.

C.14 Cost estimate

The cost estimate is based on the following data:

- Schedule of rates, Land Reclamation Project - September 1985;
- Schedule of rates, Chittagong O&M Circle BWDB;
- Information from the Feni Closure Project;
- Information collected from the "market".

The costs are assumed to be valid for 1 July 1986. Inflation up to that date has been included in the rates. The assumed exchange rate for Dutch Guilders is Dfl 1.00 = Taka 13.00.



LONGITUDINAL SECTION ALONG CROSS-DAM

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.47

The cost estimate for the cross-dam is summarized in Table C.11, and is based on the assumption that the works will be executed by an (international) contractor. A summary of the cost of additional investigations, final design, project bound operating cost of BWDB, the cost of improvement of the embankment on Char Lakhi, the cost of expatriate assistance to the BWDB during construction, and maintenance cost for the cross-dam are shown in Table C.12. A more detailed breakdown of the construction cost of the cross-dam is given in Table C.13.

C.14.1 Cost of alternative dam sections and closure methods

As a part of the selection process for alternative dam sections and closure methods, preliminary cost estimates have been made for these alternatives.

C.14.1.1 Alternatives for dam sections on shoals

The earthfill cross-section with slopes 1:4 on the shoals can be replaced by a cofferdam-type construction with the same crest level and a small width, as indicated in Figure C.39a. The loads on such a cofferdam construction are of the order of 2 to 3 t/m^2, resulting in expensive structures. The possibility of overtopping and wave action require rather extensive bed protection works at the base of such structures. An earthfill dam proved to be the cheapest structure, despite the expensive slope protection required.

C.14.1.2 Alternatives for western closure

The cost of a closure-dam completely built up of concrete blocks, and the cost of a Feni-type closure, have both been estimated. According to these estimates, the Feni-type closure dam on top of the sill will be approximately Tk 70 000 000 cheaper than the closure dam built up of concrete blocks. On the other hand, the sill required for the Feni-type closure will have to be larger, require considerably more material, and cost approximately Tk 150 000 000 more than the sill for the concrete block dam. Moreover, because of the stock pile islands on top of the sill, more ballast material will be required for bed protection in the case of the Feni-type closure, at an additional cost of approximately Tk 85 000 000. Because of the longer construction period (about 4 months), another Tk 16 000 000 will have to be added for site overhead cost. Summing up, the construction costs of the Feni-type closure are estimated to be approximately Tk 180 000 000 higher than the cost of the concrete block dam, or about Tk 215 000 000 higher if general overhead cost and contingencies are also taken into account. The much greater risks involved in the Feni-type closure are not included in this figure.

Table C.11 - Summary of cost estimate cross-dam

		Taka million	Percentage of total
1.	Mobilization & Demobilization	60	2.2 %
2.	Camps	51	1.9 %
3.	Work harbours, power, stockpiles	72	2.6 %
4.	General transport, communications, work roads	41	1.5 %
5.	Loading facilities mainland	24	0.9 %
6.	Bed protection middle	20	0.7 %
7.	Bed protection west	253	9.1 %
8.	Bed protection east	430	15.4 %
9.	Earthfill dam middle	53	1.9 %
10.	Closures middle + final dam	54	1.9 %
11.	Earthfill dam west (km 0-3.75)	184	6.6 %
12.	Closure west + final dam (km 3.75-5.25)	426	15.3 %
13.	Earthfill dam east (km 14-19.2)	247	8.9 %
14.	Closure east + final dam (km 19.2-22)	750	26.9 %
15.	Site clearance	5	0.2 %
16.	Site overhead	120	4.3 %
	Sub-total	2790	100 %
	General, overhead, risk, profit 15 %	418	
		3208	
	Contingencies approximately 10 %	322	
	Total cost in million Taka	3530	
	Equivalent cost in million Dfl.	272	

Notes: Exchange rate Dfl. 1 = Tk 13.
Price basis 1 July 1986.

Table C.12 - Additional cost (taka)

	Foreign currency equivalent	Local currency
I. Additional investigations prior to final design		
Geotechnical surveys	2 400 000	6 000 000
Quarry survey	1 050 000	1 000 000
Hydraulic measurement equipment	2 250 000	500 000
Hydraulic model tests	3 300 000	1 000 000
	9 000 000	8 500 000
II. Cost of final design	25 000 000	
III. Construction-related operating cost BWDB during execution (4 years)		
Office costs		2 800 000
Salary costs		16 000 000
Transport costs		1 700 000
		20 500 000
IV. Foreign assistance during construction	100 000 000	
V. Cost of improvement of embankment on the mainland		5 000 000
Grand Total	134 000 000	34 000 000
VI. Maintenance of cross-dam Tk 600 000 per year.		

Table C.13 - Detailed breakdown of cost estimate

	Quantity	Unit	Rate Taka	Total cost Taka
Western section (km 0 - 5.25)				
Bed protection				
0.2 t/m² bricks	67 500	m²	280	18 900 000
0.4 t/m² bricks	78 000	m²	410	32 000 000
0.4 t/m² boulders	42 500	m²	650	27 600 000
1.0 t/m² boulders	135 750	m²	1250	169 700 000
Bank protection	4 000	m²	1150	4 600 000
				252 800 000
Shoals (km 0 - 3.75)				
Bags filled with local fill (gully)	11 400	m³	1200	13 700 000
Bags filled with local fill (bund)	13 400	m³	1000	13 400 000
Earthfill	480 000	m³	70	33 600 000
Slope protection	28 000	m²	280	7 800 000
Bullah piling 1 = 2 m	7 600	m²	700	5 300 000
Tarja matting on 0.5 m clay	57 000	m²	165	9 400 000
Concrete blocks on filter fabric	119 000	m²	850	101 200 000
				184 400 000
Channel (km 3.75 - 5.25)				
Sill				
Stone rubble	68 000	t	900	61 200 000
Concrete cubes	64 000	t	1100	70 400 000
Washing-in with brick chips	18 000	t	580	10 400 000
Closure-dam				
Concrete cubes up to PD + 4.50 m	150 000	t	1100	165 000 000
Concrete cubes top	83 000	t	1150	95 500 000
Washing-in with brick chips	41 000	t	580	23 800 000
				426 300 000
Total for western section				863 500 000

(continued)

Table C.13 (continued)

	Quantity	Unit	Rate Taka	Total cost Taka
Eastern section (km 14 - 22)				
Bed protection				
0.2 t/m² bricks	120 000	m²	280	33 600 000
0.4 t/m² bricks	80 000	m²	410	32 800 000
0.5 t/m² boulders	153 000	m²	740	113 200 000
Over 0.9 t/m² boulders	208 000	m²	1150	239 200 000
Bank protection	10 000	m²	1150	11 500 000
				430 300 000
Shoals (km 14 - 19.2)				
Bags filled with local fill (bund)	20 800	m³	1000	20 800 000
Earthfill	590 000	m³	75	44 300 000
Stone rubble (bund)	10 000	t	750	7 500 000
Slope protection mattress 1250 m'	35 000	m²	280	9 800 000
Bullah piling	10 500	m'	700	7 400 000
Tarja matting on 0.5 m clay (4000 m')	66 000	m²	165	10 900 000
Concrete blocks on filter fabric	172 000	m²	850	146 200 000
				246 800 000
Channel (km 19.2 - 22)				
Sill				
Stone rubble	120 000	t	900	108 000 000
Cubes 0.75 m	216 000	t	1150	248 400 000
Washing-in with brick chips	50 000	t	580	29 000 000
Closure-dam				
Cubes 0.75 m/1.00 m	293 000	t	1150	337 000 000
Washing-in with brick chips	48 000	t	580	27 800 000
				750 200 000
Total for eastern section				1 427 300 000

(continued)

Table C.13 (continued)

	Quantity	Unit	Rate Taka	Total cost Taka
Central section (km 5.25 - 14 km)				
Bed protection				
Shoals	36 000	m²	280	10 100 000
Gullies	15 000	m²	650	9 800 000
				19 900 000
Chars and shoals				
Earthfill	852 000	m³	35	29 800 000
0.3 m clay + turfing	237 000	m²	90	21 300 000
0.5 m clay + matting	9 000	m²	165	1 500 000
				52 600 000
Central closures (400 m and 600 m)				
Timber jetty	400	m'	8300	3 300 000
Bags filled with clayey earth	20 000	m³	1200	24 000 000
Earthfill	359 000	m³	45	16 200 000
0.3 m clay + turfing	48 000	m³	90	4 300 000
0.5 m clay + matting + bricks	31 000	m³	190	5 900 000
				53 700 000
Total for central section				126 200 000

Notes: Price basis 1 July 1986.
Exchange rate Dfl. 1 = Taka 13.

C.14.1.3 Alternatives for eastern closure

The cost of three alternative closure methods for the eastern closure have been estimated. Closure by means of caissons on top of a sill of stone rubble and concrete cubes is about 10 % more expensive than the preferred closure-dam made completely of stone rubble and concrete cubes.

The cost of a closure by means of a steel jetty with flapgates on top of a sill and an earthfill final dam profile is about 11 % higher than the preferred solution. This alternative was already eliminated in the pre-selection process.

C.14.2 Cost of alternative construction materials

Approximately 44 % of the total construction costs is related to the supply of sand, shingles, boulders, stone rubble and cement. A part of these materials is required for the slope protection with concrete blocks for the earthfill dams across the shoals. The cost of these concrete block protection works are about 55 % of the total cost of these dam-sections.

Alternative slope protections are more expensive. A sand-asphalt slope protection is 4 % more expensive. A slope protection consisting of gabions filled with bricks is 50 % more expensive. The latter cost is very sensitive to the type of corrosion protection on the gabion wire mesh.

Most of the materials are required for making 367 000 m^3 of concrete cubes of various sizes. Cheaper alternatives are possible if suitable rock is found (in large quantities) in Bangladesh. The availability of solid rock with a specific weight of about 2.6 t/m^3 in a "nearby" location is a prerequisite for this. Importing stone rubble from abroad could be an economical solution, but implies more foreign exchange expenditure. The cost of imported stone rubble is of the same order as local rock presently obtained from small quarries.

Another possibility to decrease the cost is to replace stone rubble and the 0.5 m concrete cubes (about 30 % of the total) by 1 m^3 gabions filled with bricks. Model tests will have to prove the feasibility of this replacement. The possible savings are about Tk 200 per tonne.

C.14.3 Cost of heavy equipment

Construction of the closure sections requires the use of much water-borne equipment, such as stone-dumping vessels, barges and tugboats. Heavy earthmoving equipment is required for the earthfill dams on the shoals to obtain the necessary high implementation capacity. Backhoes and cranes are required to handle the heaviest stone rubble and concrete cubes. An indicative list of equipment is given in Table C.14. A part of this equipment is available in Bangladesh. However, the major part has to be imported.

Table C.14 - Indicative list of equipment

Item	Number required
Trucks 10 t	8
Trucks/dumpers 30 t (17 m^3)	20
Bulldozers (D7 or equivalent)	3
Grader	1
Wheel loaders 3 m^3	3
Excavators 2 m^3	2
Cranes 2/4 t	2
Tractors	5
Flat wagons	5
Block transport wagons 0.3 t	15
Concrete batching plant 35 m^3/h	2
Concrete block-making machines	2
Transport barges (for mainland to site transport) 1250 t	2
Work boats 100 hp	4
Stone dumping barges 600 t	4
Tugs 400 - 600 hp	4
Flat pontoons 500 t/300 t	4
Water carrier barge 800 t	1
Water carrier trucks	2
Fuel carrier trucks	2
Power generators	2000 kW

The length of the execution period is not such that the equipment can be completely depreciated on the project. As the equipment required cannot be used economically for the usual type of projects in Bangladesh, it makes sense to leave it to the contractor to import this equipment on a temporary basis. Discussions with local contractors revealed that this is also the case with the earthmoving equipment. The use of local trucks with 4 m^3 capacity for the earthfill dams on the shoals would in theory be cheaper, but would increase the work period considerably, resulting in substantial additional cost. The additional cost for a delay of one year, which is unavoidable when local trucks are used for the earthfill dams, is estimated at about Tk 110 million.

C.15 Execution aspects

C.15.1 General

The fastest execution method is often also the cheapest, mainly in view of the considerable overhead cost for an extended construction period. Another important factor is the required flexibility with regard to changes in the location and dimensions of the flow channels. The design of the closure works and the related work methods should allow for sufficient flexibility, so that any change after the final design or even after the start of construction works can be dealt with. The closure method recommended for the two major flow channels (dams of stone rubble and concrete cubes) fulfil these requirements. The required heavy equipment can only be used economically if it can be used for various parts of the work.

The work components can be separated into two main groups: water-borne operations and land-based operations. The first group consists of the bed protection in the gullies, the sills, possibly the heavy elements of the top layers of the closure dams, and the transport of most materials over water. The second group includes the dam construction on the islands and the actual construction of the closure-dams in the main gullies (with possible exception of the outer protective layers). The elements on the shoals take an intermediate position.

C.15.2 Work plan and time schedule

The main challenge of the construction of the Sandwip cross-dam is the purchase and transport to the site(s) of huge quantities of materials within a relatively short time. Table C.15 shows an estimate of the total quantities involved, resulting in an average delivery quantum of 2000 t per day during a (minimum) work period of about three and a half years.

Apart from the materials to be incorporated in the works, substantial amounts of consumables, such as fuel and drinking water for labourers, will have to be carried to the site(s).

A proper organisation of the logistics is of primary importance for the successful implementation of the cross-dam project.

A time schedule for the works is given in Figure C.48. The schedule is the shortest possible and would impose the highest demands on the contractor(s). Yet the schedule is not unrealistic. Key aspects for the execution are:

- Earthfill operations should be carried out between the beginning of December and the middle of March;
- Slope protection works on earthfill dams should be finished before the beginning of May;
- Construction works for the sills should not start too soon after the end of the monsoon season;
- The central channels should be closed in the first full winter season;
- The western and eastern closures should be realised in subsequent seasons;
- Land-based equipment required for earthfill works can also be used for the construction of the rockfill dams.

TIME SCHEDULE FOR IMPLEMENTATION OF CROSS-DAM

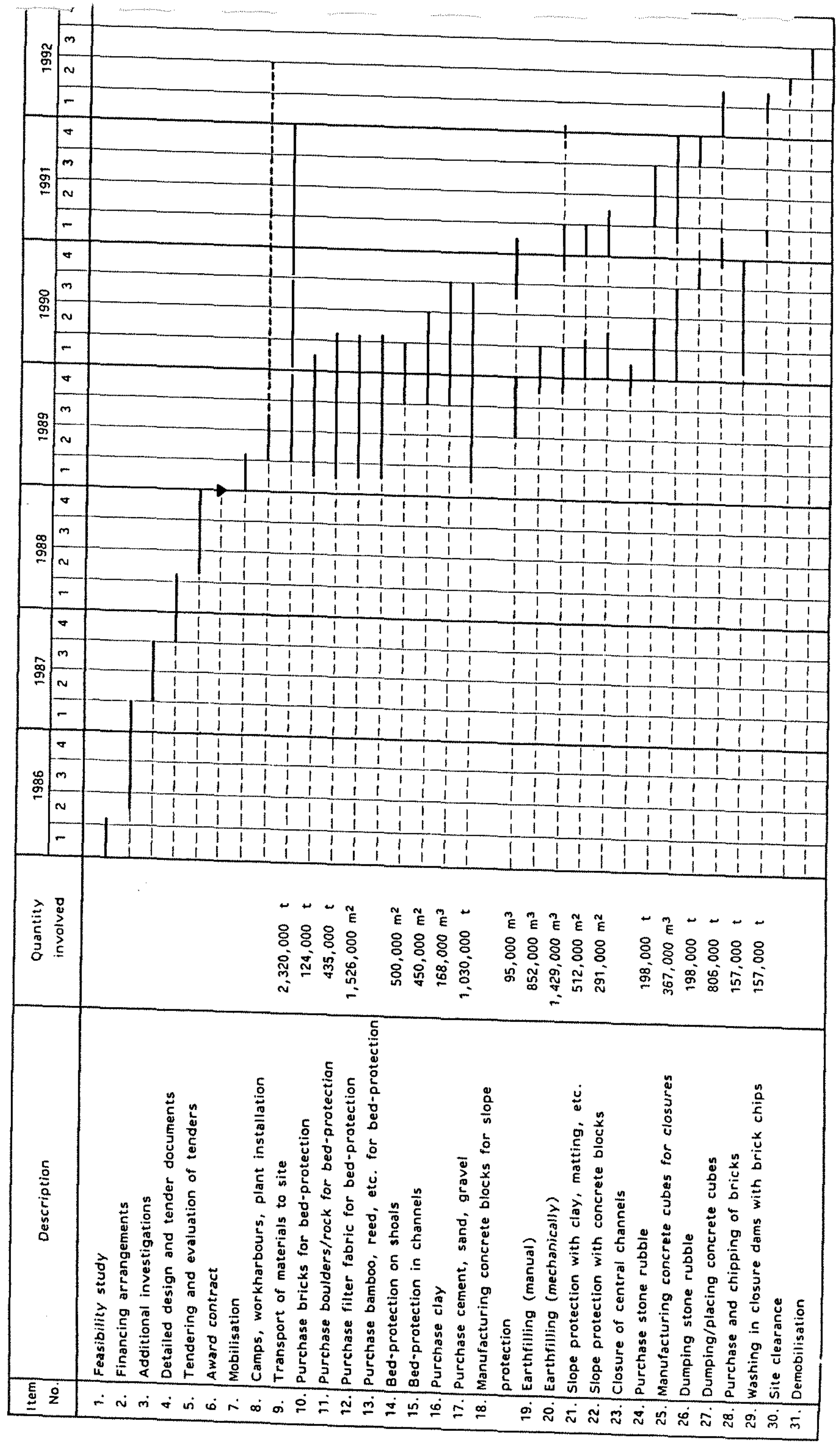


Figure C 48 Time Schedule

Table C.15 - Quantities of materials for entire cross-dam

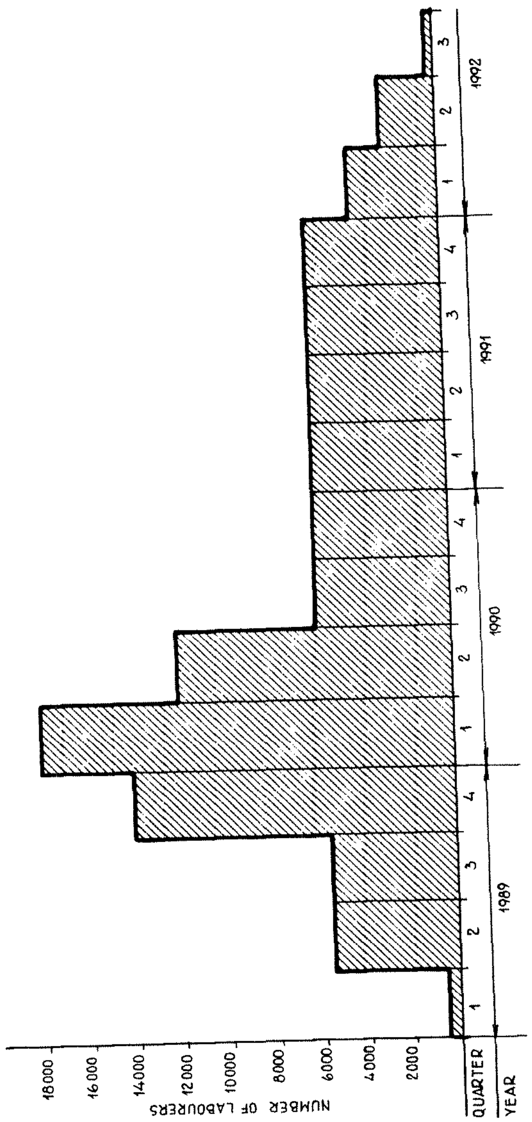
	Central	Western	Eastern	Total
Land acquisition		20 ha		
Earthfill	1 210 000 m^3	480 000 m^3	590 000 m^3	2 280 000 m^3
Clay	106 000 m^3	29 000 m^2	33 000 m^3	168 000 m^3
Jute bags (no.)	800 000	1 000 000	840 000	2 640 000
Bullah piles 8 m	8 000 m^1	-	-	8 000 m^1
Bamboo 6 m (no.)	98 000	132 000	277 000	507 000
Reed bundles 0.3 m dia. (no.)	25 000	252 000	417 000	694 000
Filter fabric bed protection	56 000 m^2	450 000 m^2	700 000 m^2	1 206 000 m^2
Bricks	10 000 t	51 000 t	63 000 t	124 000 t
Boulders 0.3 dia.	6 000 t	17 000 t	77 000 t	100 000 t
Boulders 0.3 dia.	-	139 000 t	196 000 t	335 000 t
Wire netting	36 t	260 t	240 t	536 t
Tarja matting	40 000 m^2	57 000 m^2	66 000 m^2	163 000 m^2
Turfing	285 000 m^2	-	-	285 000 m^2
Filter fabric slopes	-	131 000 m^2	189 000 m^2	320 000 m^2
Concrete for blocks (slopes)	-	40 000 m^3	55 000 m^3	95 000 m^3
Bullah piles 2.50 m dia. 0.15 m	-	113 000 m^1	158 000 m^1	271 000 m^1
Stone rubble	-	68 000 t	130 000 t	198 000 t
Concrete in blocks (sill + cl.dam)	-	135 000 m^3	232 000 m^3	367 000 m^3
Brick chips	-	59 000 t	98 000 t	157 000 t
Cement	-	41 000 t	66 000 t	107 000 t
Sand FM 2.5	-	61 000 t	100 000 t	161 000 t
Sand FM 1.8	-	61 000 t	100 000 t	161 000 t
Shingles	-	228 000 t	373 000 t	601 000 t
Total weight mat. excl. fill	230 000 t	790 000 t	1 300 000 t	2 320 000 t

Large-scale use of labour for some of the main work items, such as earthfill operations (for dams on shoals), construction of sills, and construction of closure dams (concrete blocks), is not possible for physical reasons: the earth filling progress rate would be too low, or the elements for sill and closure dams would be too heavy. Nevertheless there will be abundant opportunities for the employment of labourers in many components of the works. It is estimated that eight million mandays will be required in various activities. This figure does not include the labour related to boulder collection, brickmaking, etc., but only reflects the work on site. The estimated average numbers of labourers for each quarter, required during the construction period, are shown in Figure C.49. Based on experience in Bangladesh, the recruiting and managing of these numbers, generally supplied by labour-contractors, will not be a problem.

As mentioned above, the time schedule given in Figure C.48 is the shortest possible, but not unrealistic. On the other hand, the BWDB has stated that: "Procurement of materials might be a great problem. Huge quantities of construction materials, some of which have to be imported, have to be procured and this is likely to affect the construction programme of other sectors at national level" and concludes that: "The time schedule for implementation appears to be too tight".

One of the advantages of maintaining the shortest possible construction time is that the risks of damage to the uncompleted works and delays caused by heavy weather or sea conditions are kept to a minimum. The fastest execution period is often also the cheapest, mainly in view of the considerable extra overhead cost for an extended construction period. Another advantage is that the benefits expected to be generated by the new polders on the accreted land appear earlier. In Annex G, Section G.5.2 the results of some sensitivity analyses have been presented, including cases in which the construction period for the cross-dam is extended from 3½ to 5 and 7 years. As is to be expected, a longer construction period yields a lower EIRR, but the effect on the economic feasibility of the project is not significant.

Selection of construction methods, whether by (international) contractors or otherwise, selection procedures, construction time schedules etc. can be worked out in detail and in close consultation with the BWDB during the final design and tender preparation phase of the project. Within the scope of this feasibility study it may suffice to show that the economic feasibility of the project is not sensitive to a doubling of the construction period, from 3½ to 7 years.



LABOUR REQUIREMENTS FOR DAM CONSTRUCTION

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. C.49

C.15.3 Phased execution of the works

It has been assumed that the whole cross-dam will be implemented. From a purely technical point of view preference is to be given to an implementation of the whole dam in a construction time which is as short as possible. This can only be achieved if one single contractor is entrusted with the construction works. The main reasons for these preferences are that costs and risks can be minimised, and that changes during construction, possibly necessary as a result of changing site circumstances, can be made smoothly.

Should there however be pressing reasons, such as the desire to contract parts of the works separately, or to implement the western and eastern closure with a considerable time interval, then this would be possible, but at a cost. Indicative (time and cost) estimates have been made for the following scenarios:

(a) Contracting the closure of the central channels and the dam section on Char Pir Baksh separately

The interrelations between closures is very strong. Preparation work for the western closure would have to start ahead of the actual closure of the central closures. If the central closures cannot be completed in time, the "western" contractor would have to wait one full year for a next attempt at the central closures.

In view of the scope of the central closures, it is unlikely that experienced international contractors in closure works will be interested in a separate contract for the central closures.

As a result of the above, a time interval of at least one year, and more likely two years, should be programmed in the case of a separate contract for the central closures.

The direct extra cost will be somewhat (but not very much) higher in case of separate contracting (assuming present values for the cost of the work). The risks for the entire project will increase in view of the extended construction time.

(b) Contracting western and eastern closure separately (5-10 years time interval)

(b1) Sequence West-East

As already pointed out in Section C.4 a closure of the western channel hardly influences the flow (discharge, velocity, etc) through the eastern channel. Though the central channels did not form part of the hydraulic model, the influence of the western closure on the flow through the central channels can be expected to be larger than on the eastern channel. It is advisable to close the central channels prior to the western channel (for which scenario (a) could be used if desired).

The cost of mobilisation, site preparation, site overhead cost, and less efficient use of equipment will weigh more heavily on the cost of each separate closure, when compared with contracting the western and eastern closures jointly. If it is assumed that the eastern channel (as a whole) will not change much after the western closure, the direct extra costs for separate contracting are expected to be in the region of Taka 150-200 million (apart from any adjustments for inflation).

In this scenario the erosion pattern of Sandwip (with a present erosion rate of 300 m per year) will not be significantly influenced by the western closure.

(b2) Sequence East-West

The eastern closure should be preceded by the central closures, for which scenario (a) could be followed. The extra direct cost for separate contracts for the western and eastern closure is the same as for scenario (b1): Taka 150 - 200 million.

As an earlier eastern closure significantly increases the flow (discharge, velocity) through the western channel, a substantial adjustment (wider and deeper) of the western channel has to be expected. This implies extra costs for the closure. It is at this stage not possible to predict a new equilibrium channel configuration for the western channel for that situation. As such it is not possible to verify (at this stage) whether the developed closure strategy would still apply. Any cost estimate for a closure in the new situation would be very speculative, but it can be said that the extra cost for a later (5-10 years) western closure will be substantial.

In scenario (b2) the erosion at Sandwip (N-W coast) will be halted, but erosion at Char Pir Baksh will no doubt accelerate.

C.15.4 Postponement of execution

The time schedule for implementation of the cross-dam presented in Figure C.48 is based on the assumption that arrangements for the financing of the construction would be finalized in the first quarter of 1987, that additional investigations and preparation of final design and tender documents would be completed in the first quarter of 1988 and that, after tendering, a contract for construction could be awarded at the end of 1988 so that actual implementation could start at the beginning of 1989. Although it would still be possible to schedule the afore-mentioned activities in such a way that awarding of the contract could take place around 1 January 1989, it is probably more realistic now to reckon with a delay of at least one year. It has also been suggested that the construction of the cross-dam might be even further postponed, for a non-specified number of years. Some remarks concerning the consequences of such a postponement are made below.

The most important result of postponing the construction would be that the erosion of the north-western coast of Sandwip island (and of the western coast of Char Pir Baksh) would continue at more or less the present rate of 400 ha per year. There is no reason to believe

that the erosion will decrease to zero. As long as the channels between Sandwip Channel and Hatia Channel are not closed, the shifting of these channels will continue and the erosion rate may fluctuate over the years, but basically will remain constant.

This means that not only the loss of valuable land, but also the pauperization of the peasants in the affected areas will continue. It also means that the length of the cross-dam to close the channels will increase at a rate of about 400 m per year, and the cost of construction will increase accordingly. Consequently, postponement of the construction of the cross-dam will have a relatively small but negative effect on the economic viability of the project.

C.15.5 Selection of contractors

Closure works are very special construction projects, whose execution requires a contractor with substantial experience. Many components of the works have to be executed under daily changing circumstances. Apart from the daily changes also considerable seasonal changes have to be dealt with. For a successful implementation of the closure-dam, it is of extreme importance that optimum use be made of the favourable conditions, while the unfavourable conditions should be avoided as much as possible. Despite all possible precautions, it will be necessary to improvise when unexpected situations arise. Only a contractor with experience in similar marine works and with a true understanding of the tidal mechanism determining the working conditions on the site, can be expected to complete the Sandwip cross-dam successfully and on time.

For that reason it will be recommendable to pre-select the contractors who will be invited to tender on the basis of proven experience in similar works. Other usual pre-selection criteria, such as financial capability and work experience in the region (preferably in Bangladesh) should of course apply as well.

It is expected that contractors with experience in comparable works or contractors experienced in (rubble-mound) breakwater construction, reinforced with external expertise if necessary, could be pre-qualified for the construction of the cross-dam. Pre-qualification criteria should be drawn up as soon as the implementation scheme for the cross-dam has been decided upon.

Evidently, the selection of contractors has to follow the procedures in force in Bangladesh and should be in accordance with the guidelines and agreements between the government of Bangladesh and the donors involved.

C.15.6 Risk sharing

Almost every civil engineering contract involves certain risks. It is a generally accepted practice to define in a contract which risks are for the account of the contractor and which are for the owner.

For closure works an important area of risks is the changing channel bed configuration. During construction the total net cross-section almost always increases, but it is difficult to predict by what percentage. Certain allowances have been made in the cost estimates for extra quantities of materials required in the works to compensate for deeper or wider channels.

In civil engineering contracts it is customary to pay the contractor for the actual volume of work done, provided he is doing it in the most efficient or acceptable manner. This practice ensures that the contractor does not have to incorporate a very high contingency percentage in his offered price for the works, which should cover all (or most) possible changes in the channel bed configuration. For the Sandwip cross-dam it is proposed that re-measurement provisions be included in the contract, so that the contractor is paid for a theoretical amount of work, which is based on surveys made at well-defined moments and/or intervals and on the theoretical minimum acceptable cross-sections. The timing of the surveys should be such that:

- the actual channel bed configuration shortly before the start of the works (or part thereof) is reasonably represented by the surveys, and
- the contractor is forced to adopt a working schedule which minimises the quantities of materials.

All the surveys are to be made jointly.

C.16 Summary of conclusions and recommendations

(a) The alignment of the cross-dam as assumed in the pre-feasibility study is adequate, except for the location of the eastern closure-dam, which should be shifted 1.5-2 km to the South.

(b) The recommended sequence of closures, based on technical requirements, is: the central channels first, then the western channel, and lastly the eastern channel.

(c) Earthfill dams with protective revetments are recommended on the shoals.

(d) Gradual vertical rock/concrete blocks closures are recommended for closure of the western and eastern gullies.

(e) The central channels can be closed with methods used before in Bangladesh.

(f) The final dam profile may be overtopped occasionally (10-15 times per year), except on Char Pir Baksh.

(g) The profiles of the rock closure-dams can be converted into final dam sections with minor modifications: the crest levels have to be raised and the dimensions of the blocks in the rock closure-dam have to be increased (compared with the situation in which the blocks are only dimensioned for closure purposes).

(h) The cost of the entire cross-dam has been estimated at Tk 350 crore (3500 million), assuming an integrated implementation. The approximate percentages for various components of the dam are:

- dam on Char Pir Baksh, including central closures (km 5.25 - 14) : 5 %
- western closure (km 0 - 5.25) : 35 %
- eastern closure (km 14 - 22) : 60 %

(i) The minimum construction time is 3.5 years, calculated from the moment of award of contract, provided the contract is awarded in the proper season. Later awarding will increase the construction time by one year.

(j) Additional hydraulic field measurements should be carried out prior to the final design stage; these measurements are detailed in Appendix C.III.

(k) Hydraulic model investigations are necessary for an optimum design of the stone closure-dams; they are detailed in Appendix C.IV.

(l) Geological investigations/surveys should be implemented with a view to locating (preferably large) rock deposits in the Chittagong Hill Tracts. If suitable rock is found in large quantities the cost of the works can be significantly reduced.

(m) Additional geotechnical investigations are necessary in the alignment of the cross-dam, particularly in the western and eastern channels. The deeper parts, in which the final closures will be made, deserve the closest scrutiny. The investigations involve both field and laboratory tests.

(n) A pre-qualification exercise should ensure that only contractors with adequate experience in similar works are invited to tender for the works.

APPENDICES

APPENDIX C.I
DESCRIPTION OF THE NETFLOW MODEL

Problem area

The computer programme NETFLOW has been developed as a powerful tool, applicable to a wide range of problems concerning the dynamic flow of water in a network of channels. Typical applications are:
- propagation of flood waves
- evaluation of flood control measures
- design of channel systems
- operation of reservoirs and irrigation systems
- water management in polder areas
- studies in tidal areas and estuaries.

In addition NETFLOW can be used in combination with programme modules for specific problem areas where NETFLOW provides the water movement. Thus the applicability of NETFLOW includes also:
- sediment transport in rivers or channel systems
- salt intrusion in surface water systems
- water quality.

Physical background

A mathematical model for a NETFLOW simulation of a channel flow consists of elements such as:
— channel reaches each identified by a name and each having its own system of co-ordinates;
— junctions connecting the reaches;
— cross-sections describing the storage and flow profile as a function of height; they may be inserted anywhere along channel reaches while the programme determines (by interpolation) sections where it needs them;
— boundary conditions such as water levels, discharges;
— structures or local head losses at any place in the reaches; and
— lateral discharges and inflows, also anywhere along reaches.

Figure 1 illustrates the definition of the various model components.

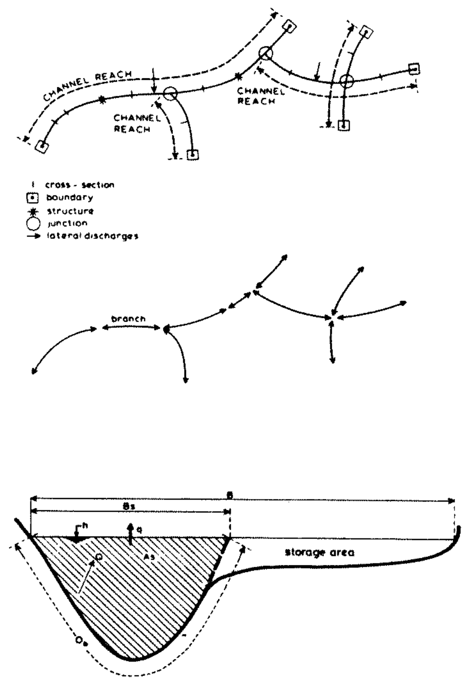


Fig. 1 Definition sketch for model components.

In agreement with the definition of junctions and structures, the programme divides the channel reaches into sections called branches. The flow in the branches is described by the St. Venant equations for non-steady flow.
Equation of continuity:

$$B_s \frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} + q = 0$$

Momentum equation:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(\alpha\, Q^2/A_s) + g\, A_s \frac{\partial h}{\partial x} - B_s \frac{\tau_s}{\varrho} + O_b \frac{\tau_b}{\varrho}$$
$$+ g\, A_s \eta = 0$$

Where τ_s and τ_b are the shear stress coefficients at the water surface and bottom respectively, η is the local head loss coefficient and α is a coefficient to account for a non-uniform flow distribution.

The significance of the other symbols is illustrated in Figure 1.

Junctions, structures and boundary conditions are defined by special equations relating Q, h and t at these locations:
— junctions: continuity in water surface and discharges;
— boundaries: F (Q, h, t) = 0 with Q and h being the discharge and the water level at the end of a branch; and
— structures: F $(Q_1, Q_2, h_1, h_2, t) = 0$ with the indexes 1 and 2 indicating the sides of the structures.

In general loss of momentum at junctions of channels is not taken into account as it is very difficult to define these losses accurately. Therefore at junctions continuity in water level and discharge is normally assumed. Nevertheless, the programme NETFLOW provides special facilities to take into account energy losses at specific locations if desired, such as losses at bridges, structures or at junctions.

If the effect of wind on the water movement has to be accounted for, the wind shear stress coefficient τ_s in the momentum equation is defined as:

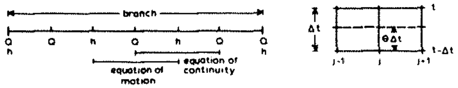
$$\tau_s = \varrho_a C_s \gamma V_w^2 \cos \phi$$

where:
ϱ_a density of air (about 1.25)
C_s friction coefficient (about 0.003 for $V_w > 16$ m/s)
γ shielding effect (influence of high banks, buildings, etc.)
V_w wind speed at a standard height of 10 m
ϕ angle between wind direction and the direction of the current.

In addition the bottom shear stress term $O_b \frac{\tau_b}{\varrho}$ is modified in order to account for the return flow of water along the bottom of the system.

Numerical solution

The above partial differential equations are written in an implicit finite difference form, defined on the following staggered grid (Θ = implicity factor):



Under normal circumstances, the complete set of non-linear difference equations in Q and h is linearized every time step. If necessary, for instance in case of very irregular cross-sections or strongly non-linear boundary conditions, the equations are handled by a Newton iteration. In both cases the resulting set of linear equations is solved by a direct technique.
The dimension of the resulting large matrix is reduced by preceding elimination of internal branch equations and by substitution of h in Q.

Boundary conditions

At each end of a channel reach which is not connected to other reaches the user has to specify a boundary condition. In general a boundary condition defines the water level or the discharge at the boundary as a function of time. Complicated boundary conditions such as dam-break waves or tidal sluices can be defined in user-written Fortran functions, which are evaluated at each computation step in order to arrive at a linear equation in h and Q. The possibility to use the unknowns h and Q and a number of computed variables in the function description makes this facility extremely flexible.

In case of a discontinuous boundary condition, such as the operation of regulating weirs, the boundary condition can be specified for various 'phases'. The expression has to be continuous during the phase; selection of the appropriate phase is done by the programme on a user-specified criterion for Q, h or t.

Special facilities

The NETFLOW system possesses a variety of facilities which makes it a very user-friendly package allowing the user to meet exactly the computational requirements of his specific study. The most important facilities are:
— description of structures and boundary conditions by user-written Fortran functions
— treatment of channels that become dry during computation or alternatively inundation of dry channels or flood plains
— Engelund procedure to account for a non-uniform roughness and velocity distribution in the cross section
— option to select steady flow computation
— preprocessing of model data by a special programme.

In particular the latter facility makes it possible to define a model by mainly entering the available field data. The time-consuming schematization, for instance specification of cross-sections at all computation points, is done by the programme itself. Thus preparation and adaptation of models can be realised with minor effort.

Input and Output

With NETFLOW the construction of the model and the simulation of the water movement can be realised in separate runs. The so-called 'model-run' pre-processes all the data relevant for the schematization of the model so that they can be efficiently used in one or more subsequent 'process-runs'.
Input to a model run comprises:
— names and lengths of channel reaches
— position of junctions (if any)
— dimension and location of cross-sections
— nature and description of boundary conditions
— specification of the bottom stress
— description of desired result functions
— definition of initial situation.

The output of a 'model run' consists of a description of the model, plots of network layout and cross-sections and the input files for subsequent process runs.
A process run requires the following input:
— start time, time step and end time
— desired distance step Δx between computation points
— parameters for the selection of computational options
— additional information describing printer output and/or plotter output
— Fortran functions describing boundary conditions and special features representing the specific situation to be simulated.

Output of a process run consists of:
water levels and discharges at desired locations as a function of time or at desired moments as a function of a spatial coordinate. The result functions are printed and optionally plotted.

APPENDIX C.II
DESCRIPTION OF THE DELFLO/DELQUA MODEL

Introduction

The design of engineering works in rivers, estuaries and coastal waters requires a thorough knowledge of existing and future flow conditions. This is essential in order to safeguard the environment, to be able to compute the strength of structures or to choose the most suitable location for water intakes or outlets. In addition, the local and general flow patterns or the quality of the water may change to such a degree as a result of the works proposed that supplementary action needs be taken.
In particular the morphology of the river and seabed is sensitive to variations in the flow conditions. Therefore, in order to ensure that a proper decision is made, it is necessary to assess the influence of the proposed activities quantitatively.
Hydraulic studies are therefore needed and in many cases a numerical model is an indispensable tool. It can often be the only tool, for instance when the area to be investigated is too large for a hydraulic scale model or when the available time to perform the study is short. With the increased capacities of computers, mathematical models can be, and are, used for an ever increasing range of problems.
The DELFLO/DELQUA program package has been developed for the modelling of water flow and transport of dissolved matter. The program originates from a package which was built by the RAND Corporation [1]. The program package has been developed further by the Delft Hydraulics Laboratory (DHL) in close collaboration with the Data Processing Division of the Dutch Department of Transport and Public Works.

The mathematical model

The approach which is used to model the water flow is based on what is referred to as the two-dimensional shallow water assumption. In this case the equations of motion have been simplified by the assumption that the pressure is hydrostatic. Depthwise variations of the horizontal velocity components being neglected, the water motion can be adequately described by three quantities: the water level and two (depth-averaged) horizontal velocity components. If the flow in the area is well-mixed vertically, these three quantities are also sufficient to describe the transport of dissolved matter, for example heat or effluents.

These assumptions and simplifications impose restrictions on the applicability of the approach.

There is, however, a wide range of problems which can be successfully tackled by this approach. Generally the flow conditions in estuaries and coastal seas or river sections, with or without tidal flow, satisfy the characteristics of shallow water flow, and in addition when tidal velocities are greater than, say, 0.5 m/s, a vertically well mixed situation is usually achieved.

Governing equations

The differential equations describing the two-dimensional horizontal flow are the shallow water equations:

$$(1)\quad \frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + V\frac{\partial U}{\partial y} - fV + g\frac{\partial \zeta}{\partial x} - k\left(\frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 U}{\partial y^2}\right) + g\frac{h+\zeta}{2\varrho}\frac{\partial \varrho}{\partial x} + g\frac{U(U^2+V^2)^{1/2}}{C^2(h+\zeta)} - \frac{1}{\varrho(h+\zeta)}\tau_x^s = 0$$

$$(2)\quad \frac{\partial V}{\partial t} + U\frac{\partial V}{\partial x} + V\frac{\partial V}{\partial y} + fU + g\frac{\partial \zeta}{\partial y} - k\left(\frac{\partial^2 V}{\partial x^2} + \frac{\partial^2 V}{\partial y^2}\right) + g\frac{h+\zeta}{2\varrho}\frac{\partial \varrho}{\partial y} + g\frac{V(U^2+V^2)^{1/2}}{C^2(h+\zeta)} - \frac{1}{\varrho(h+\zeta)}\tau_y^s = 0$$

$$(3)\quad \frac{\partial \zeta}{\partial t} + \frac{\partial((h+\zeta)U)}{\partial x} + \frac{\partial((h+\zeta)V)}{\partial y} = S$$

where
U = depth averaged velocity in x direction
V = depth averaged velocity in y direction
ζ = water elevation relative to the reference plane (see Fig. 1)
h = distance from the bottom to the reference plane (see Fig. 1)
τ_x^s, τ_y^s = wind stress components in x- and y-direction
f = Coriolis parameter
g = acceleration due to gravity
k = horizontal momentum diffusion coefficient
ϱ = density of the water
C = Chézy parameter for bottom friction
S = source or sink term

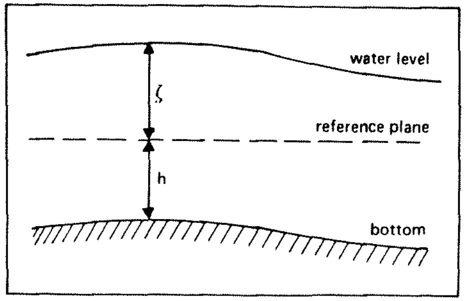


Fig. 1 Definition of water elevation (ζ) and bottom depth (h).

The term $g\frac{h+\zeta}{2\varrho}\frac{\partial\varrho}{\partial x}$ in Equation (1) and the corresponding term in Equation (2) take into account the forces due to lateral density gradients which can be caused for example by horizontal gradients in the salinity distribution.

The mass balance equation for a vector $\bar{P}$ of constituents reads, for vertically well-mixed flows:

$$(4)\quad \frac{\partial((h+\zeta)\bar{P})}{\partial t} + \frac{\partial((h+\zeta)U\bar{P})}{\partial x} + \frac{\partial((h+\zeta)V\bar{P})}{\partial y} - \frac{\partial((h+\zeta)D_x\frac{\partial\bar{P}}{\partial x})}{\partial x} - \frac{\partial((h+\zeta)D_y\frac{\partial\bar{P}}{\partial y})}{\partial y} + [K](h+\zeta)\bar{P} = \bar{S}$$

where

$\bar{P}$ = the vector containing all constituents to be studied in the problem

[K] = the matrix containing first order linear reaction coefficients

$\bar{S}$ = the vector representing source and sink terms for the constituents

D_x, D_y = the dispersion coefficients in x- and y-direction.

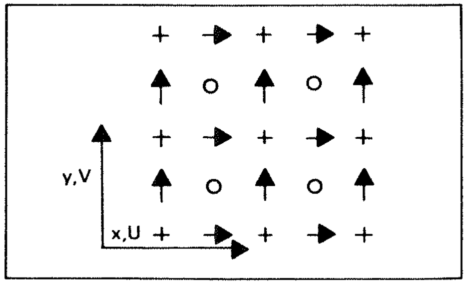
Numerical solution

In the numerical method used in this program, the area under consideration is covered by a two-dimensional mesh of square grids (Fig. 2).
Recently the numerical solution has been extended to cover the case of curvilinear orthogonal grids of variable size.
Equations (1), (2) and (3) are solved by an implicit finite difference method (ADI). This method is chosen because it allows large time-steps without any negative effects on the computational stability. Grid-staggering is employed which gives a satisfactory degree of accuracy without incurring excessive computer costs.
Water elevation and velocity components are computed at all grid points and used in the mass balance equation for the constituents (4), which is also solved by an implicit finite difference method.

If the constituents are passive in the sense that they do not induce density differences which influence the water movements, the mass balance equation (4) can be solved separately.

In this case the velocities and water elevations are simply used as inputs for the mass balance equation, which can then be solved with space and time steps which may be different from the ones used in solving Equations (1), (2) and (3). This uncoupling is imperative if, for example, the reaction processes have time scales which are much larger than those of the water movements. This separate mass balance solution can in a number of cases be obtained by the water quality part of DELFLO, referred to as DELQUA.

In other cases offline coupling with dedicated water quality models like DELWAQ is possible. This model allows elements of arbitrary geometry and user-supplied descriptions of the kinetics of the constituents to be employed. With these options, almost any type of water quality problems can be handled.
Another important coupling is that with the WAMOR program package, dealing with sediment transport and morphological developments. Here the same spatial discretisation is used as in the DELFLO model providing the data on water movement.
A complete description of the mathematical background of the DELFLO program is given in [1] and [2].



+ : water elevation and constituent point (ζ, P)
○ : depth point (h)
→ : velocity component in x-direction (U)
↑ : velocity component in y-direction (V)

Fig. 2 Grid used in the finite difference scheme.

Special facilities

The DELFLO system pocesses a number of facilities for simulating physical phenomena and the effects of hydraulic structures. The most important are:
- a depth-averaged k-ε model, which can be coupled in cases when a detailed knowledge of the turbulent momentum diffusion coefficient is needed
- several procedures for the simulation of flooding and drying of intertidal flats
- a facility to simulate weirs and sluices whose flow characteristics can be varied with time
- the possibility to represent narrow dams like groynes and breakwaters
- the possibility to simulate the discharge of heat and effluents and the intake of cooling water at any location in the computational grid
- (offline) nesting of models.

Input data

The input data necessary to run the model are:
a. for water movement:
- geometry and bathymetry of the area;
- velocities and/or water levels at the open boundaries;
- locations and flow characteristics of sluices, barriers, intakes, outfalls;
- wind velocities (if important);
- roughness parameters;
- momentum diffusion coefficient

b. for the transport of constituents:
- boundary conditions at open boundaries;
- concentration of constituents at outfalls;
- equilibrium temperature of receiving water, or solar radiation, convection, evaporation etc., if the model is used for cooling water studies;
- dispersion coefficients;
- reaction coefficients if constituents are non-conservative.

Output

The model computes water levels, velocities and constituent concentrations at every time step in all grid points. These results can be displayed in various forms:
- time history graphs of water level, velocities and constituent concentrations;
- two-dimensional velocity vector plots;
- cotidal plots;
- isolines of water level, velocity and concentration of constituents;
- three-dimensional plots of water level (and bathymetry).

Quantities like transport through cross sections and residual flows can also be computed and displayed.

APPENDIX C.III
ADDITIONAL HYDRAULIC MEASUREMENTS

1. The bathymetric changes in the channel-shoal-island system between Noakhali and Sandwip have to be followed through regular surveys (2 to 3 months intervals).
2. Simultaneous discharge measurements have to be made at several points in the channels (west, central, east).
3. A wave rider buoy should be installed (and verified regularly) for wave measurements during a whole year.
4. Velocity measurements should be made covering a complete series of tidal cycles between spring and neap tide, with particular emphasis on the occurrence of multiple slack waters.
5. In addition to the above measurements, data available on cyclones and their effects should be collected from various sources. A study performed by BIWTA may provide valuable data. These data would give more accurate information on surge heights at the boundaries of the mathematical models; these data are also indispensable for the assessment of wind effects within the model boundaries.

APPENDIX C.IV
MODEL INVESTIGATIONS

1. The design of the "stone" closure in the feasibility phase has been based on a general study in which initiation of damage was investigated. For a final design it is recommended that some model tests be performed in which typical dam sections are simulated in situations with damage. Based on such a study the amount of spare blocks can be determined. This study can be done in Bangladesh at the River Research Institute under the guidance of experienced researchers.
2. The length of the bed protection should be checked for scouring by a model investigation. The criterion to be used in this investigation is the non-undermining of the edge of the bed protection. These tests can also be done in modelling facilities available in Bangladesh.

   If the geotechnical survey indicates that thick loosely packed sand layers are present, the scour investigation must be extended to more flow situations during the closing operations in order to determine the total maximum scour depth. In case of flow slides, the length of the bed protection is directly related to the total maximum scour depth.
3. One of the alternative materials for rock in the sill/closure dam is gabions filled with bricks. For a proper design their stability should be checked by a model investigation, both for current and wave attack. Large model facilities are necessary for this investigation; it can only be performed by a restricted number of institutions in the world, among which the Delft Hydraulics Laboratory in the Netherlands.

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ANNEX D
MORPHOLOGICAL
EFFECTS

ANNEX D - MORPHOLOGICAL EFFECTS

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ANNEX D
MORPHOLOGICAL EFFECTS

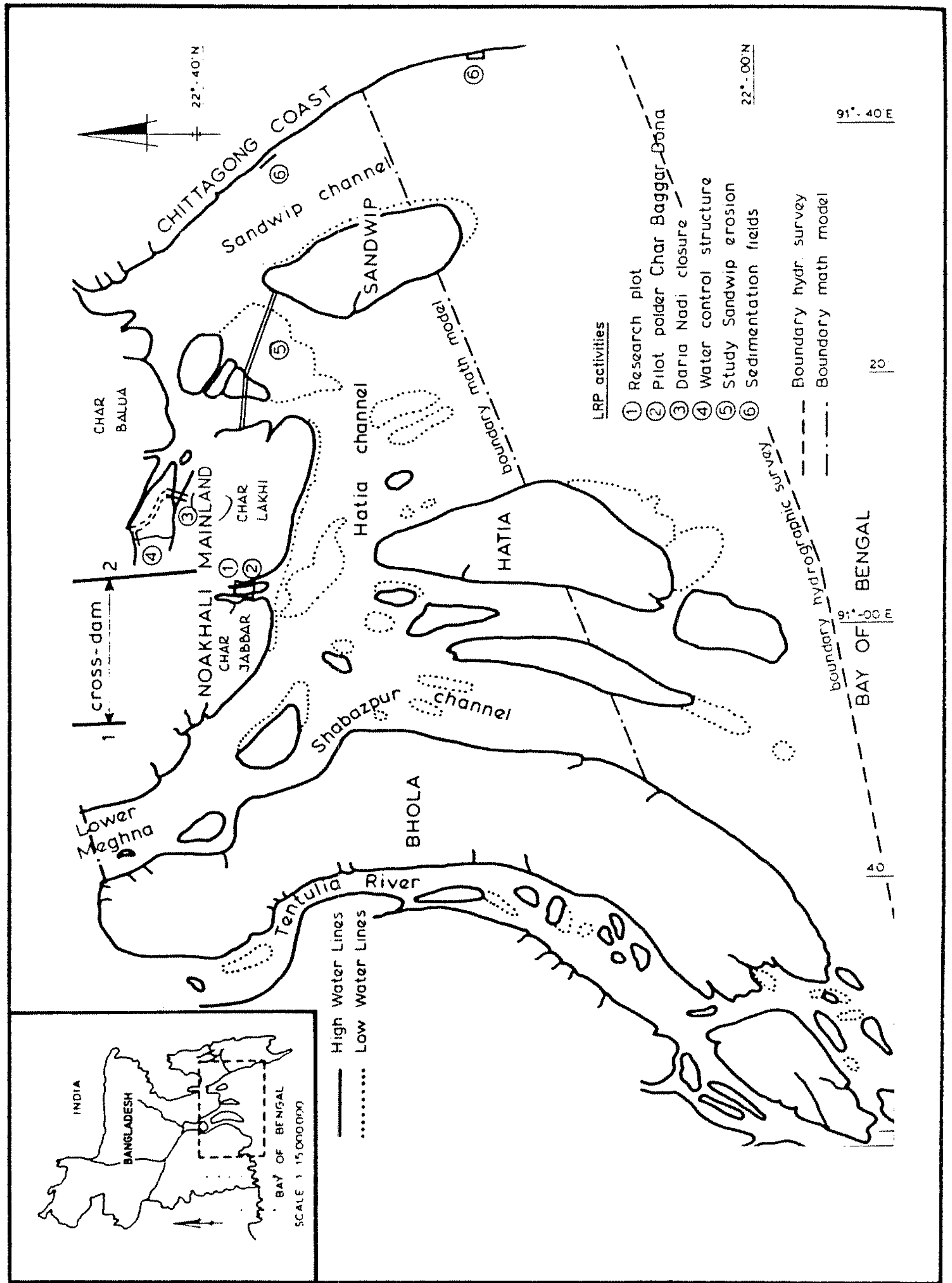
D.1 Introduction

The pre-feasibility study on measures to protect Sandwip and Hatia from erosion, carried out in 1983/1984 (LRP, 1984a), concluded amongst other things, that the best method to protect Sandwip island is to construct a cross-dam between Sandwip and the Noakhali mainland. Such a dam would not only halt this erosion, but would also lead to the subsequent accretion of an estimated area of 22 000 ha new land which could be reclaimed and developed in due course.

This conclusion was drawn from computations with a two-dimensional model, based on the computer program WAQUA, which had been developed for the eastern part of the estuarine area. The model covers an area reaching from the east coast of Bhola to the Chittagong coast, while its southern boundary runs about halfway across Hatia and Sandwip (Figure D.1). After calibration of the model on the basis of available prototype surveys and measurements, computations were made for various situations with regard to the alignment and construction phases of the cross-dam, and for the situation corresponding with the recommended alignment, after the expected accretion (LRP, 1984b).

Another conclusion drawn from these computations was that the cross-dam and subsequent accretion near the dam have a local effect. No significant effect could be observed in the Shabazpur channel, while the current velocities along the north coast of Hatia appeared to remain virtually the same in all situations. Therefore, it could be concluded that the construction of the cross-dam and the subsequent accretion will have no significant effect on the erosion of Bhola, nor on the erosion of the north coast of Hatia.

Mathematical models have also been used in the present feasibility study, amongst others to study the morphological effects of the construction of the cross-dam more closely. The overall two-dimensional model used in 1986 covered only the eastern part of the original model used in the prefeasibility study, with the western boundary running from the north coast of Hatia to Char Jabbar on the mainland. This was considered sufficient because the original model had shown that the effects in the western part are negligible. In addition, two detail models have been developed near the western and eastern channel, on both sides of Char Pir Baksh (see Annex C, Section C.2.3.2). The computations with these models have been used, amongst others, to predict the accretion near the cross-dam. Moreover, a one-dimensional model based on the computer program NETFLOW has been developed and used in the feasibility study, in which the geometry of the area is represented by a network of channels which connect storage basins (see Section C.2.3.1).



PROJECT AREA WITH SURVEY AND MODEL BOUNDARIES

SCALE 1:≈716,000

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG D.1

Because of the concern expressed by various agencies in Bangladesh, after the submission of the draft feasibility report, with regard to possible adverse effects of the construction of the cross-dam, and the subsequent accretion on, amongst other things, the Outer Anchorage of Chittagong Port, the shipping route between Chittagong and Dhaka (notably Hatia Channel), and the erosion of the north coast of Hatia and other locations in the area, additional computations were made in 1987 with the one-dimensional model for the present situation, the situation upon completion of the cross-dam, and the situation after the expected accretion near the dam.

The results of the computations of the above-mentioned mathematical model with regard to the morphological effects of the construction of the cross-dam are discussed in Sections D.4, D.5 and D.6.

It has been suggested that in addition to the mathematical models, the use of a physical hydraulic model covering the entire south-eastern delta should also be considered. Apart from the technical studies, such a model could be used to demonstrate, visualize and explain the various aspects and consequences of the construction of the cross-dam.

It should be understood that it is not possible to simulate tidal currents, density currents, sediment transport and waves, which all play a role in the morphological processes, in one and the same physical model, because of the different scale requirements for the reproduction of these phenomena. The primary objective of a physical model, therefore, would be to determine water-levels and currents at different conditions and various locations. As in the feasibility study using mathematical models, morphological studies would have to be performed separately by means of computational methods using current data determined in the physical model.

The construction of a physical hydraulic model for the entire eastern delta would require considerable funds and time. A preliminary estimate indicates an initial investment in the order of Tk 40 million, and a period of minimal two years to design, construct, and calibrate the model. Moreover, frequent reconstruction of the model would be necessary to take into account the morphological changes in the delta, in order to determine the resulting changes in the water movement which, in turn, will be the input for subsequent morphological studies. In general, similar and even better results can be obtained from mathematical models at a fraction of the cost of physical overall models and within considerably shorter periods. Consequently, the construction of a physical model for the eastern delta as envisaged can not be justified from a technical point of view.

On the other hand, it may be considered to improve the predictions and impact assessment of the construction of the cross-dam, and the accretion by computations with two-dimensional flow and sedimentary transport models during the detailed design phase of the project. This two-dimensional model should cover the whole LRP area, and the southern open-sea boundary of this model should coincide with the seaward boundary of the general, one-dimensional model, so that the model can also indicate possible morphological impacts on the southern tip of Sandwip island.

D.2 Methodology

Generally, the cross-sections of tidal channels in a loose-sediment environment show a clear relationship with the tidal prism passing that cross-section if tidal motion dominates (Bruun, 1978; Salomons and Eysink, undated; Eysink, 1979; Eysink, 1983a). Similar relationships have been shown to exist for other relevant quantities, such as the total channel volume and the relative channel area in a tidal flat area (Salomons and Eysink, undated). These types of relationships, combined with the computed changes in the water movement due to the construction of the cross-dam, have been used in the present study to predict the ultimate changes in the morphology of the estuary. The application of the relative changes in water-levels and flow velocities or discharge rates is a reliable method of judging whether the Sandwip cross-dam will make a significant impact on the local morphology and, if so, to what extent. This is the only way to determine the limits beyond which there will be no significant impact from the cross-dam. Then the changes, as before, would be caused only by the intense dynamics of the Lower Meghna estuary itself, demonstrated by the continual shifting of the channels due to the absence of any bank protection.

In the channels to be closed by the cross-dam, the tidal volumes will reduce visibly. Consequently, the channels will undergo siltation to adapt to the changed hydraulic conditions, resulting in accretion of land near the cross-dam. Rough estimates have been made of the time involved for the adaptation process, based on the total volumes of sediment required for the adaptation, and the future sediment transport patterns which must supply those volumes.

On the tidal flats a different approach had to be used, based on the settling of sediment from the water column above the bed, resulting in a logarithmic bed-level rise. The coefficient in the formula was checked with field data on the siltation related to the Feni and Daria nadi closures. Basic data for the investigations of morphological aspects were collected from the various site surveys performed under the Land Reclamation Project.

D.3 Present situation

D.3.1 General characteristics of the area

The project area is situated between Sandwip island and the Noakhali mainland, and is part of the huge estuary of the Lower Meghna river. The two main channels which will be closed by the cross-dam are secondary tidal channels connecting Sandwip channel and Hatia channel in the north with the emerging new island, Char Pir Baksh, in between.

Char Pir Baksh actually consists of four relatively high chars covered with grass, some mangrove forest, and with rice paddies on the highest char, which are inhabited in the higher parts above PD + 4.4 m. The houses are built on small mounds about 1 m high, as a protection against inundations during the monsoon season. As was recently proved, this

protection is unfortunately inadequate against inundations during severe cyclones.

These chars are separated by three relatively small channels, two of which run more or less south to north. These so-called central channnels will also be closed by the cross-dam.

At present, both the western side of Char Pir Baksh and the north-western coast of Sandwip island are being eroded by the rather strong currents flowing through the two main channels connecting Sandwip channel and Hatia channel. At the same time accretion is taking place in the channel between the mainland and Char Pir Baksh, and on the eastern side of Char Pir Baksh. The present morphological processes occurring near Char Pir Baksh appear to result in maintaining but shifting of the cross-channels in an eastern direction.

However, the not yet stabilized and productive land emerging on the western side of these channels may ultimately be eroded once more. The two so-called central channels through Char Pir Baksh mentioned above, which appear to be increasing in size, and another channel, now developing along the coast of Char Lakhi are indications of this phenomenon.

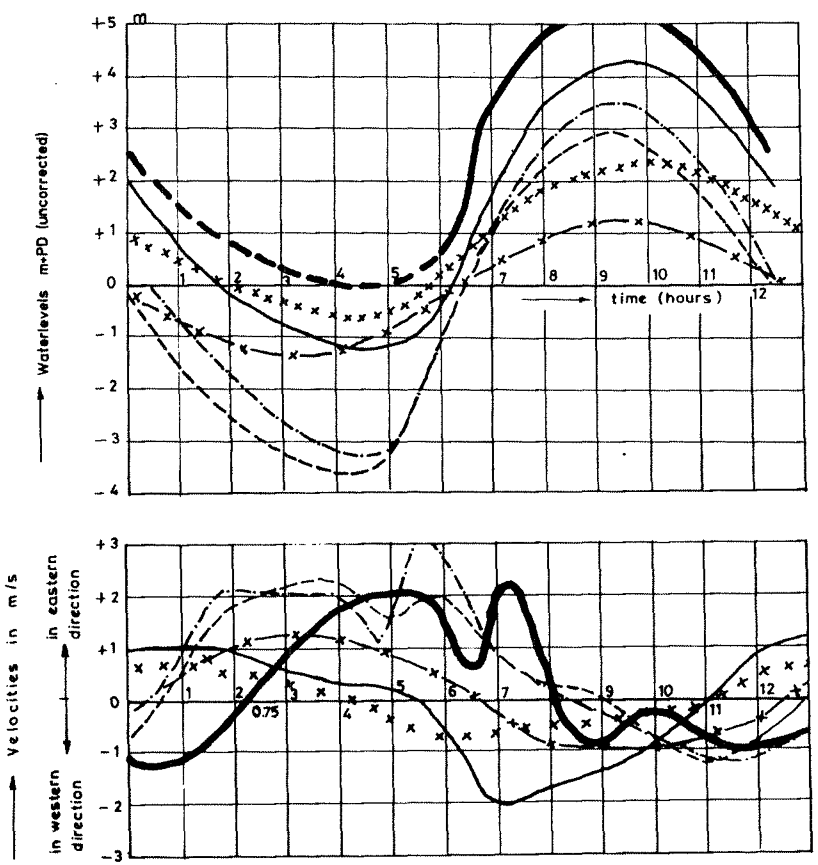
About 9000 ha of new land already lies above high water, outside the existing mainland embankments. It may be expected that the accretion in front of Char Jabbar, Char Bata and Char Clark will continue for some time, and an additional area estimated at about 3000 ha may emerge above high-water.

The land on Char Pir Baksh, which at present lies above high-water, covers an area of about 6000 ha. As mentioned above, this land is rapidly being eroded on its western side, at a rate of about 300 m per year. Without the construction of the cross-dam, investments for infrastructural improvement and development of this area cannot be justified.

D.3.2 Water movement around Char Pir Baksh

The tide approaches the project area through Sandwip and Hatia channels from the south. The tidal waves propagating through these channels first meet in the channel, north of Sandwip island, and later in the other channel, north of Char Pir Baksh. Since the tidal wave through Sandwip channel reaches the secondary channels slightly earlier than the one through Hatia channel, due to differences in the distortion of both tidal waves, the water-levels at the ends of the secondary channels are not generally the same, resulting in tidal exchange flows through the channels. This phenomenon explains the existence of the two channels.

The flow through both channels is dominated by the tide. The maximum flow velocities occur around local high and low water in the direction of Hatia channel and Sandwip channel respectively (Figure D.2). The effect of the upland river discharges on the flow velocities in the channels is only of minor importance, as is confirmed by salinity data on the estuary (Eysink, 1983b). The same principle is true for both channels running north-south through Char Pir Baksh.



——— currents during spring tide 9.3.85 in channel between Char Pir Baksh and Char Balua

– – – – currents during spring tide 9.3.85 in channel between Char Pir Baksh and Sandwip

—·—·— currents during spring tide 30.7.84 in channel between Char Pir Baksh and Sandwip

xxxxxxx currents during neap tide 16.3.85 in channel between Char Pir Baksh and Char Balua

—x—x—x currents during neap tide 16.3.85 in channel between Char Pir Baksh and Sandwip

▬▬▬ currents during spring tide 15.7.84 in channel between Char Pir Baksh and Char Lakhi

FLOW CONDITIONS IN THE CHANNELS WEST AND EAST OF CHAR PIR BAKSH

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.2

The flow velocities in the deepest part of these channels are quite strong. Under spring-tide conditions the maximum velocities in the two main channels are generally in the range of 2 - 2.5 m/s and may even rise to 3-4 m/s for a short period under extreme tidal flows.

The tidal range in the area of Char Pir Baksh is high, varying between about 2-6 m with the neap and spring tide cycle. During the monsoon season the local mean sea level rises about 0.6 to 0.75 m relative to the level in the pre-monsoon season, due to the combined effect of reduced air pressure, wind set-up, and upland river discharge (particularly the reduced salinity effect).

The wave climate in the area is modest. The waves are wind generated and mainly occur in the monsoon season when the winds are southerly. However, wave heights are then generally less then 0.5 m, and only rarely exceed a height of 2 m.

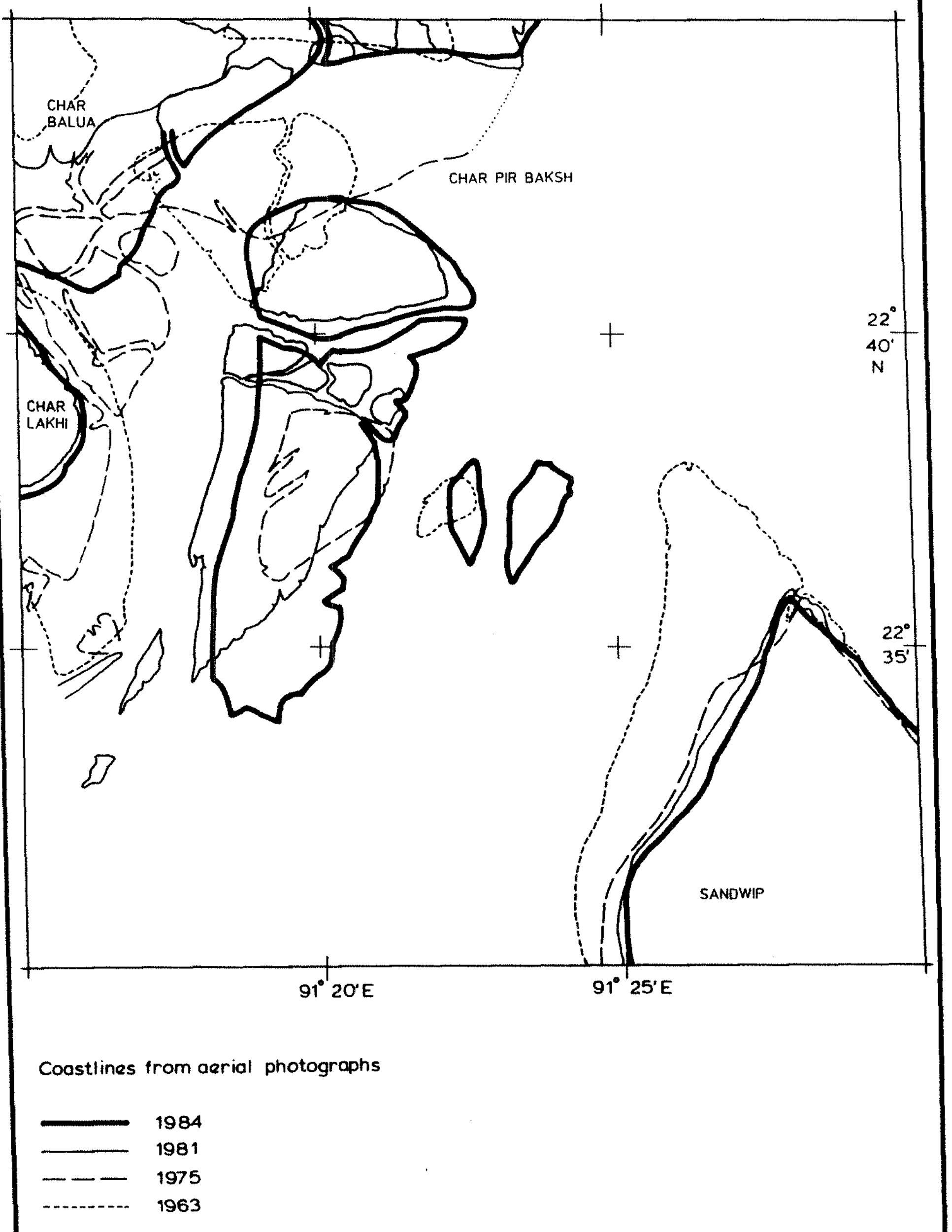
D.3.3 Sedimentology

The bottom composition varies from very fine sand and silty sand on the channel floors and the lower parts of the exposed sides of the chars, to soft mud on the lower parts of the sheltered and accreting sides of the chars. The dry parts of the chars and the land on Sandwip island consist primarily of clayey silt to silty clay. Because of the relatively low content of clay particles (smaller than 2 μm), and the various layers of fine sand and silt in the bottom of the high chars and Sandwip island, the resistance to erosion by currents and waves is low. This explains the serious erosion rates at the western and northern sides of Char Pir Baksh and Sandwip island respectively, where strong currents run along the banks. First, the sand layers are washed away, causing instability in the clayey soils on top, particularly at low water. These soils slide into the channel, where they are easy prey to the currents. Thus erosion rates of over 300 m/year occur.

Almost the entire area around Char Pir Baksh consists of reworked and recently deposited sediments, except for the northern part of Sandwip island, which may be of late pleistocene origin (Coleman, 1969). Char Lakhi, Char Balua, and Char Pir Baksh stem from only the past few decades (Figure D.3). All sediment originates from the Ganges, Brahmaputra and Meghna rivers, which annually supply some 1-2.2 billion t of suspended sediment (Bruun, 1978; Eysink, 1979 and 1983a).
The sediment contents and suspended sediment discharges of these rivers show a distinct seasonal influence and increase with increasing river discharges (see Tables D.1 and D.2). Almost the entire sediment load passing the Lower Meghna consists of very fine sand, silt and clay.

Data on suspended sediment in the estuary of the Lower Meghna, collected by the Survey and Study Division of the Land Reclamation Project between October 1978 and October 1980, yielded some general information on sedimentology in the area (Eysink, 1983b; LRP, 1982). The major conclusions of this study are:

- no distinct seasonal influence could be recognized in the sediment content of the estuary;



RECENT DEVELOPMENTS OF THE HW. CONTOUR-LINES IN THE PROJECT AREA

scale 1:150,000

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.3

- only a weak relationship between sediment content and the spring-tide neap-tide cycle was found, with a tendency to higher sediment content during spring-tides. The differences are most pronounced in the monsoon period;
- a significant increase of sediment content was observed with increasing distance below the water surface;
- the project area is within the maximum turbidity range in the Lower Meghna estuary. The annual mean depth-averaged sediment content in the waters around Char Pir Baksh is about 3000-5000 mg/l;
- the suspended sediment at Char Jabbar and Char Lakhi is mainly silt. Generally less than 10 % of the suspended material is very fine sand (> 63 µm) and the same is true of clay particles (< 2 µm). The median diameters varied between 15 and 34 µm and did not show any seasonal effect;
- the grain size distribution of the suspended sediment at Char Jabbar and Char Lakhi showed great similarity with the bottom samples taken there.

Table D.1 - Weighted mean monthly sediment contents of the Ganges, Brahmaputra and Meghna rivers (mg/l)

	Ganges Hardinge bridge 1958-1962	Brahmaputra Bahadurabad 1958-1962	Meghna Bhairab Bazar 1961-1962
January	244	292	0-12
February	194	220	0-7
March	146	285	7-21
April	149	517	5-20
May	143	777	25-40
June	340	1034	44-99
July	1024	1152	51-313
August	1628	1412	26-154
September	1621	1268	64-236
October	1751	759	0-107
November	555	435	0-6
December	291	309	0-3
Annual mean	1302	1004	33-163

Note: Sediment contents based on Coleman, 1969.

Table D.2 - Average monthly suspended sediment transport of the Ganges, Brahmaputra and Meghna rivers (10^6 t)

	Ganges Hardinge bridge 1958-1962			Brahmaputra Bahadurabad 1958-1962			Meghna Bhairab Bazar 1961-1962	
	Mean	Min.	Max.	Mean	Min.	Max.	Min.	Max.
January	2.0	1.0	2.7	4.0	2.4	5.4	-	0.02
February	1.4	0.8	2.4	2.5	1.6	3.5	-	0.01
March	0.9	0.4	1.5	3.6	2.2	5.3	0.01	0.06
April	0.8	0.4	1.2	9.7	5.4	17.2	0.01	0.06
May	0.8	0.7	1.0	36.4	13.0	82.7	0.09	1.26
June	3.9	1.2	8.7	87.6	58.9	147.0	0.39	1.40
July	48.7	31.7	81.9	121.6	104.4	167.6	0.77	7.50
August	168.8	32.7	197.8	163.3	108.8	227.2	0.45	3.70
September	156.1	106.8	241.8	117.8	72.1	177.9	1.30	4.80
October	81.6	61.4	196.4	42.9	23.2	71.4	-	2.28
November	10.5	7.1	20.3	12.1	10.0	16.7	-	0.08
December	3.2	1.0	4.3	5.4	4.0	7.4	-	0.01
Total	478.9	257.3	735.9	607.7	530.9	696.5	3.02	20.20

Notes: Sediment transport after Coleman, 1969.
Total mean annual sediment transport: 1.1 billion t.

The suspended sediment transport rate is determined by the depth and width-integrated product of the current velocity and the suspended sediment content. Thus it changes with the tide. Since the flow velocities in the area are strongly tide-dominated, the effect of upland discharge and wind are of less importance. Therefore it is believed that the sediment transport pattern around Char Pir Baksh is subject to only minor seasonal influences.

D.3.4 Characteristic morphological relations

As mentioned in Section D.2 the size of a channel can be related to a characteristic tidal prism passing it. In general this relation can be written as:

$$A_{MSL} = c.V \qquad (D.1)$$

where A_{MSL} : cross-sectional area of channel in m^2 relative to a characteristic level (e.g. MSL)
c : coefficient
V : characteristic tidal volume in million m^3.

A coefficient of about 80 generally applies for tidal areas taking the cross-sectional area below MSL, and the mean tidal volume as the characteristic tidal parameters. This is derived from situations where maximum flow normally occurs around MSL.

Data on the channels in the Lower Meghna estuary however, show a distinctly lower coefficient, which seems to be about 55-65 when applying the average of the ebb and flood volumes. This unusual coefficient might be explained by the fact that the channels are located in the area of maximum turbidity in the Lower Meghna, or by the asymmetry of the tidal wave. However, the flow conditions in the channels around Char Pir Baksh are different. In the cross-sections at the dam alignment, maximum flow occurs around high and low water, resulting in an even lower coefficient of 51, which indicates relatively small cross-sections in these particular short-cut channels. Due to the influence of the tidal storage area in the Noakhali khal-Daria nadi area, the flow conditions in the channel between Char Pir Baksh and Char Balua are closer to those in a normal tidal channel. Consequently, the coefficient is higher, i.e. 62.5.

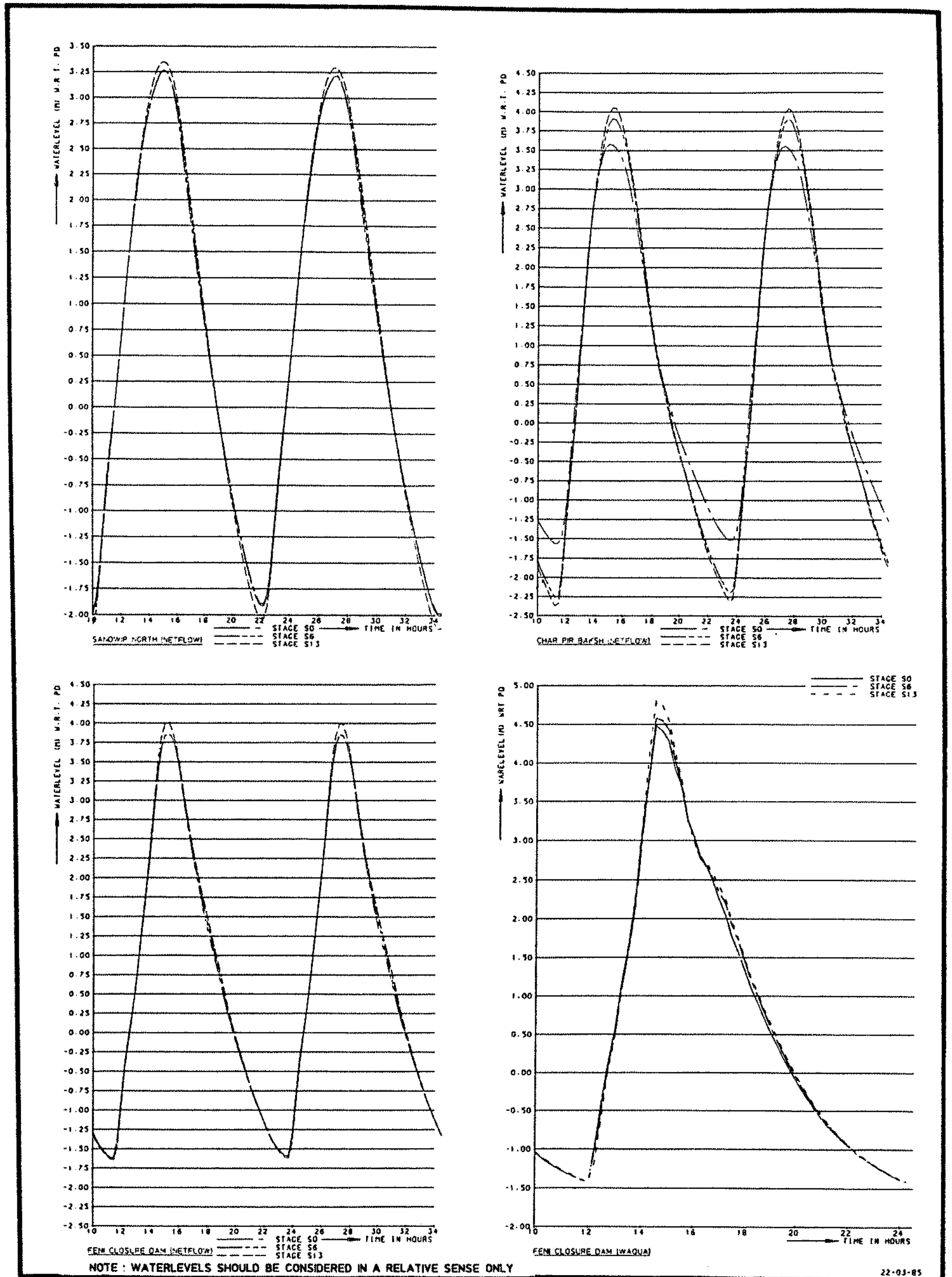
These findings, and the fact that after the closure all channels will have a normal flow pattern with maximum flow around MSL, indicate that the general relationship (Equation D.1) can then be applied relative to MSL with a coefficient of about 60, to estimate the size of the new equilibrium profiles of the channels corresponding to the tidal volumes in the new situation. The tidal flat levels, in an area which is more or less in a state of dynamic equilibrium, are known to be related to the mean high water-level.

D.4 Impacts of the cross-dam

D.4.1 Changes in tidal motion

The construction of the cross-dam will change the tidal regime in the project area, and these changes are important boundary conditions for predicting the morphological impact of the cross-dam. Mathematical models were used to determine the changes in water-levels, flow velocities and discharge rates throughout the entire Lower Meghna estuary.

The water-levels are affected most in the northern part of Sandwip channel and in the Noakhali Khal, as demonstrated by the computed changes at Char Pir Baksh and at the Feni dam. In these places the high water-levels at spring-tide will increase by 0.5 m and 0.1-0.3 m respectively, and the low water-levels will reduce by 0.8 m and 0-0.1 m respectively. At Sandwip North, the changes in high and low water-levels will only be + 0.1 m and - 0.15 m respectively, and will become negligible south of Sandwip island (Figure D.4).



WATER-LEVEL CHANGES IN SANDWIP CHANNEL	SPRINGTIDE
SANDWIP CROSS-DAM FEASIBILITY STUDY	FIG: D.4

In Hatia channel the tidal ranges will generally decrease, with the greatest changes in high and low water-levels in the western part of the cross-dam (- 0.4 m and + 0.4 m respectively at spring-tide). At Noakhali and Sandwip-Satal khal, the changes have already decreased to - 0.05 m and + 0.15 m respectively, while they become 0.05 m and less at Hatia island (Figure D.5).

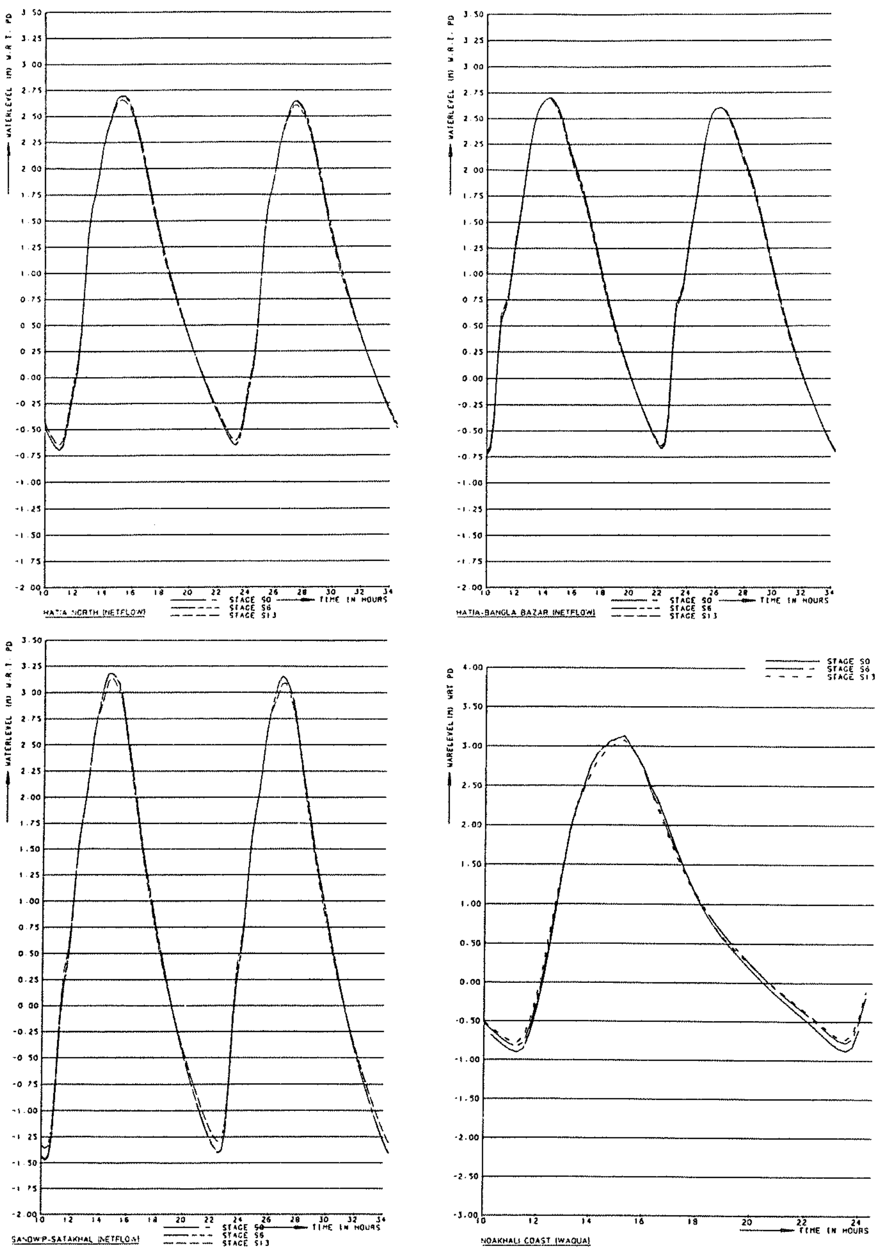
A similar picture can be drawn with respect to the flow velocities and the discharges. The major impact of the dam will of course occur in the shortcut channels running north-south, which will be closed by the cross-dam. There the flow velocities will show distinct reductions. Initially, tidal volumes will increase in Noakhali khal and Daria nadi, due to the increase of the tidal range. The increase of the mean tidal volume at the mouth of the Noakhali khal-Daria nadi is estimated at some 13 million m^3.

The flow in the channel between Char Pir Baksh and Char Balua is affected by the cross-dam in two ways. On the one hand the flow is reduced due to the blockage of the short-cut flow, while on the other hand this is largely compensated for by the extra storage near the dam and in the Noakhali khal-Daria nadi, due to the increased tidal range. It is believed that the mean tidal volume of about 60 million m^3 presently passing this channel will only be initially reduced by about 10 million m^3. The character of the flow through the channel will also change; it will lose its characteristic of maximum flow occurring close to high and low water.

The increased tidal range in the northern extremity of Sandwip channel will also result in a slight increase in the flow velocities in the channel just after completion of the cross-dam (Figure D.6). This will vanish in the long run due to the effect of accretion of new land at the dam. On the other side of Sandwip island, at Satal khal, the flow velocities will hardly change, with minor reductions in the maximum ebb and flood velocities. At Hatia North the maximum ebb velocities will reduce and the flood velocities will increase, but only by relatively small amounts (Figure D.7).

D.4.2 Effects on sediment transport patterns

The cross-dam will close the channels through which sediment is presently supplied to Sandwip channel. The sediment supply around the south of Sandwip island, however, will remain. Since the sediment content of the water in Sandwip channel and Hatia channel are about the same, the actual sediment supply through both channels to be closed is fairly modest, and in fact only occurs through the eastern channel at Sandwip North. This sediment supply is roughly estimated at 0.1-0.5 million t per tide, depending on the tide and season. Some sediment flows back again through the western channel at Char Lakhi.



NOTE : WATERLEVELS SHOULD BE CONSIDERED IN A RELATIVE SENSE ONLY

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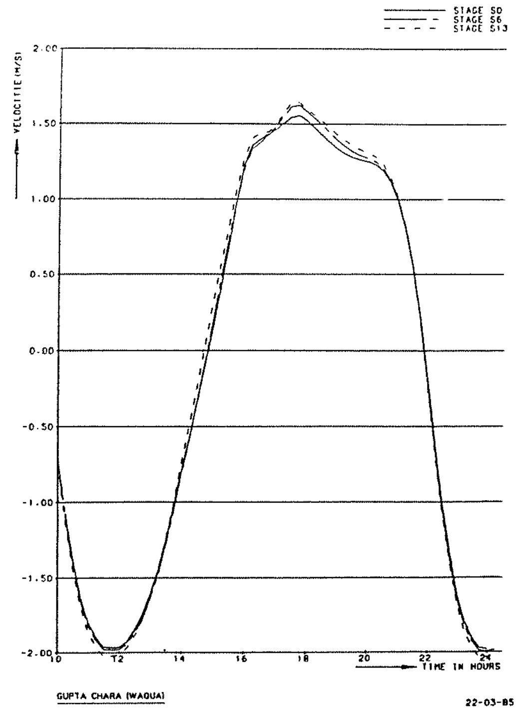
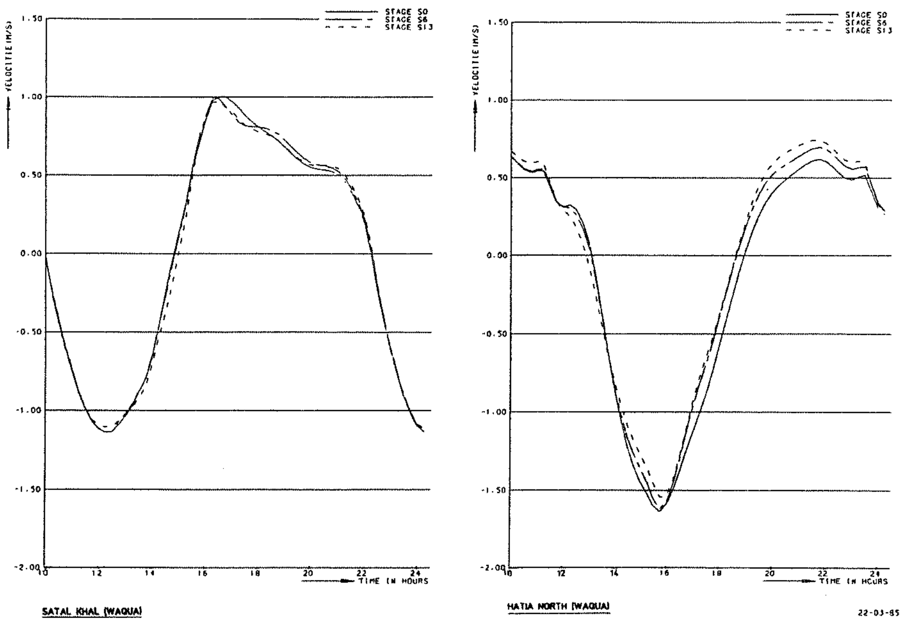
WATER-LEVEL CHANGES IN HATIA CHANNEL	SPRINGTIDE	
SANDWIP CROSS-DAM FEASIBILITY STUDY		FIG. D.5



CHANGES IN FLOW VELOCITIES IN SANDWIP CHANNEL AT GUPTA CHARA	SPRINGTIDE	
SANDWIP CROSS-DAM FEASIBILITY STUDY		FIG. D.6



CHANGES IN FLOW VELOCITIES IN HATIA CHANNEL
AT SANDWIP-SATAL KHAL AND HATIA NORTH

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.7

To demonstrate the minor importance of this source on the sediment transport in Sandwip channel, its quantity was compared with the total amount of sediment in suspension each tide around maximum flow in Sandwip channel. The latter amounts to some 50-100 million t, which is 200-500 times the supply rate. This huge amount of sediment is constantly deposited and stirred up from the bottom of the channel in a dynamic state of equilibrium. Sediment withdrawn from this 'reservoir' by net accretion (e.g. in Feni river) is regularly replaced by fresh sediment from the Ganges, Brahmaputra and Meghna rivers (mainly around the south of Sandwip island), and to a lesser extent by the Feni river. This system will not be significantly changed by the construction of the closure dam, which will cause about 7 million t/year, or only 0.01 million t each tide, to be withdrawn from the available 'sediment reservoir' of 50-100 million t. This amount will easily be provided from outside the Sandwip channel. Thus the availability of sediment should not be considered a limiting factor for accretion north of the cross-dam.

D.5 Accretion near the cross-dam

Because of the cross-dam, the flow through the north-south running channels will be blocked, and areas of very weak currents on both sides of the dam will be created. Silt-laden water penetrating into those areas will no longer be able to hold the sediment in suspension, resulting in rapid accumulation of silt. Thus, siltation of too wide channel profiles will occur, and accretion of new land will begin along the dam and the banks of the closed channels. This in turn will reduce the tidal storage area around Char Pir Baksh, causing further flow reductions and hence, further siltation of channels, shoaling of tidal flats and accretion of new land. This process will continue until a new dynamic morphological equilibrium under the modified hydraulic conditions has been achieved.

When sediment-laden water enters areas with reduced flow, part of the sediment will settle, particularly around slack water, and will not be eroded again during the next ebb or flood. It can be demonstrated that the settling rate is proportional to:

- the average silt content of the water ($\bar{c}$) in kg/m^3;
- the relative reduction of the local flow velocity squared ($\Delta u^2/u_o^2$);
- the maximum water depth (h) in m, and
- the relative inundation time (T).

This leads to the following relationship which roughly describes the shoaling rate due to the cross-dam:

$$dh/dt = -\alpha\bar{c}T(\Delta u/u_o)^2h = -\beta Th \qquad (D.2)$$

where α, β : coefficients
Δu^2 : $u_o^2 - u^2$, with u_o, u: local flow velocity without and with cross-dam respectively

This formula demonstrates that the highest shoaling rates will occur in the deep channels near the dam, where $T = 1$, $\Delta u/u_o = 1$, and h is largest.

Equation D.2 can be solved analytically for channels which are inundated throughout the year, which yields:

$$y/y_o = \exp.[-\ \beta(t-t_o)] \qquad (D.3)$$

where y : reduced water depth in m below MWL at time t in years (after a period of $t - t_o$)

y_o : initial water depth in m below MWL (time t_o)

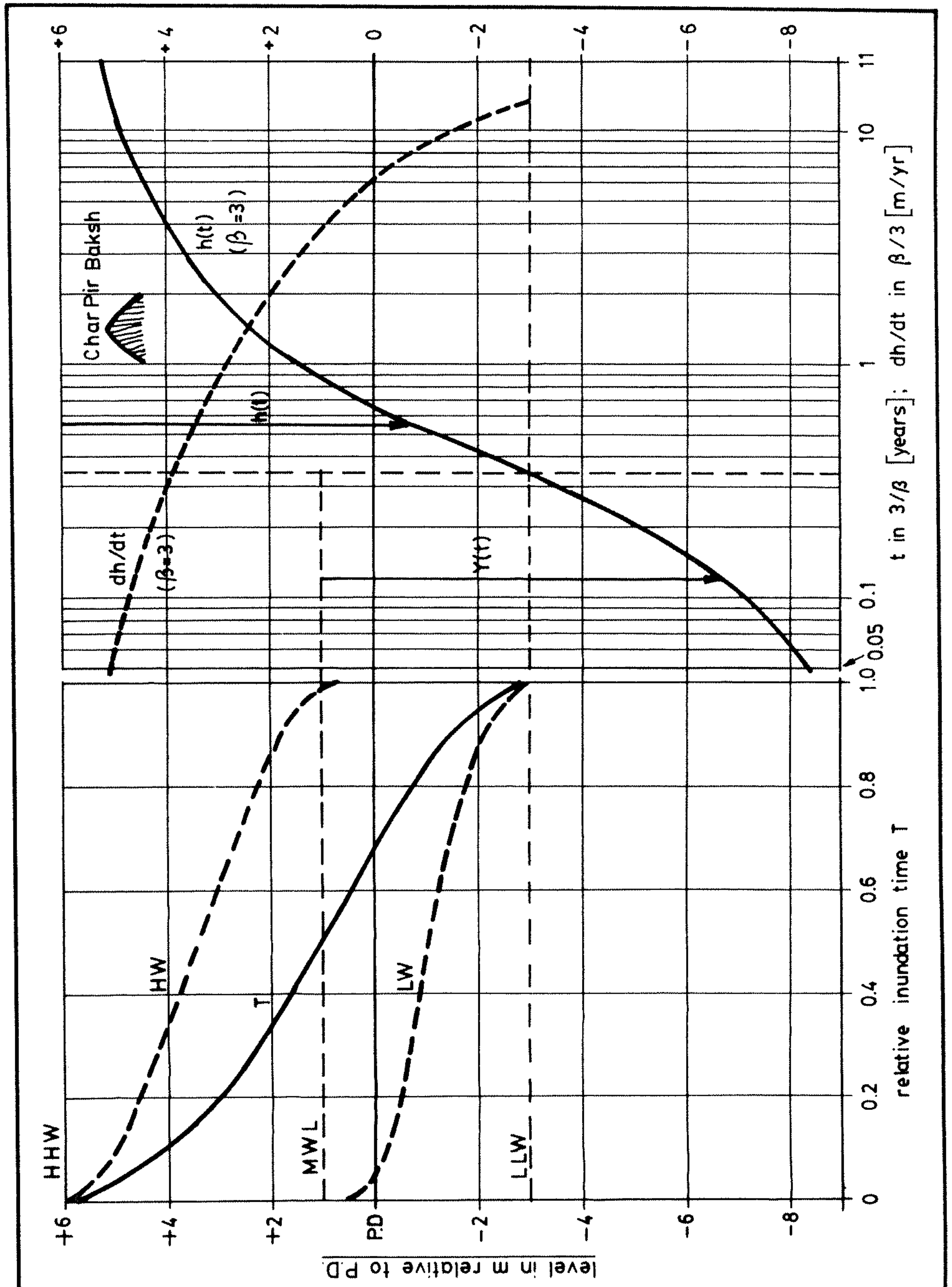
For the tidal flat area between LLW and HHW, equation D.2 must be solved numerically, taking the average inundation depth at 0.5h and the initial bed level h_o as the distance below HHW.

Assuming that 90 % of the average sediment content of the water settles permanently during each tide, then each metre in the water column would yield an annual deposit about 3 m thick. This indicates that $\alpha\bar{c} = 3\ yr^{-1}$ is a realistic first estimate for equation D.2. Therefore, this value was initially applied to plot equations D.2 and D.3 as well as the numerical solution for the tidal flats in Figure D.8.

As the next step, the coefficient was calibrated, based on shoaling data due seaward of the Feni dam and the Daria nadi closure. For the Feni dam only, tentative shoaling data are available showing a much faster shoaling process than would follow from the factor $\alpha\bar{c} = 3\ yr^{-1}$. A value of $18\ yr^{-1}$ seems to be more appropriate. The silting data at the Daria nadi dam, however, yield lower values. The initial siltation in the channel in the first two months indicates a value of $13.5\ yr^{-1}$, whereas the subsequent shoaling in the next 13 months and the shoaling of the adjacent flats over a period of 15 months indicate values of 8.5 and $9.6\ yr^{-1}$ respectively. Since the siltation conditions in the Daria nadi are believed to be more representative for the Char Pir Baksh area, and the latter two figures are the most reliable, a value of $\alpha\bar{c}=9\ yr^{-1}$ was adopted for the accretion study, or rather:

$$\beta = 9\ \Delta u^2/u_o^2\ yr^{-1} \qquad (D.4)$$

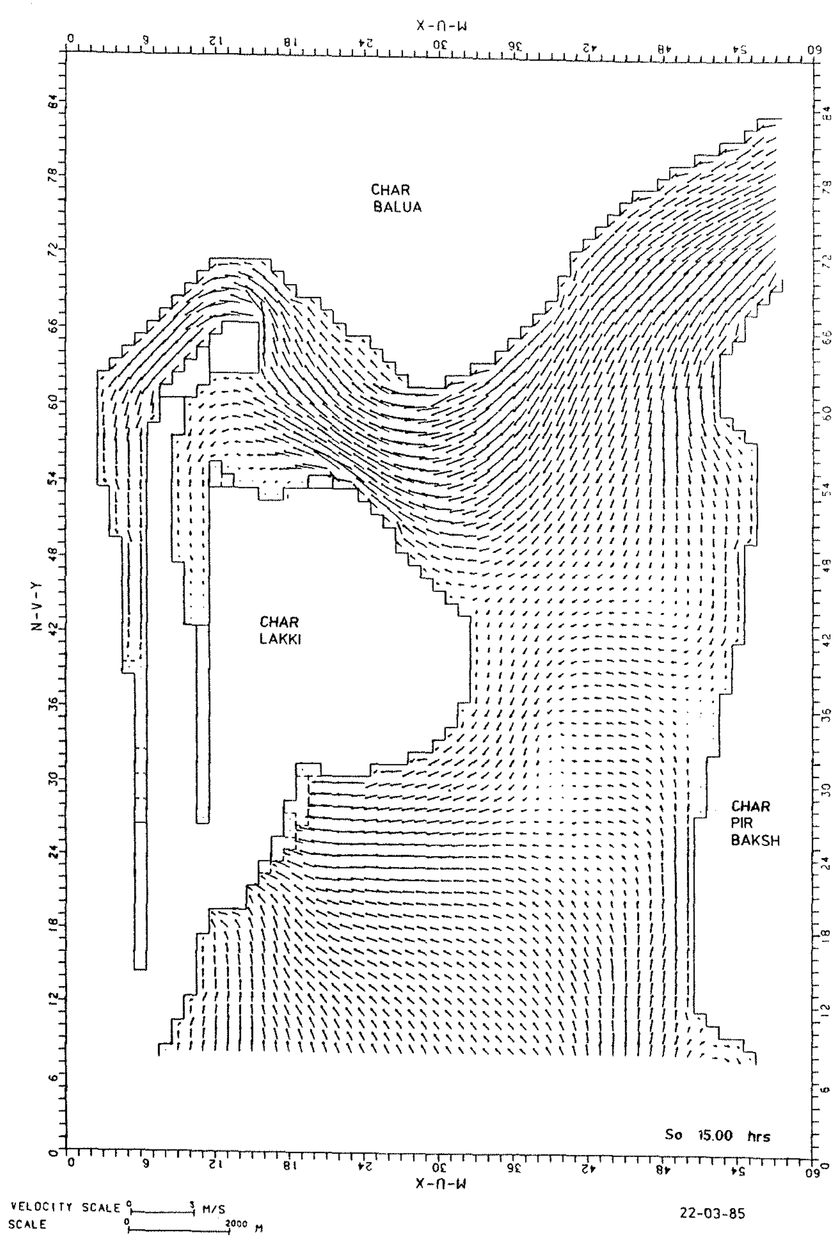
The relative flow reductions in the project area due to the construction of the cross-dam can be estimated from the computations with the detail mathematical models (WAQUA) for the situations before and after the closure. These computed results show areas with distinct flow reduction, but also areas where the flow is reduced very little, or not at all (Figures D.9 and D.10). This means that Figure D.8 cannot be used as a general basis for the prediction of the accretion history, as β is not a constant for the entire area. However, it can be used to determine the initial accretion pattern near the dam, where $\Delta u^2/u_o^2$ is close to unity. A value of τ = 1300 ha/year has been derived from Fig. D.8. The initial accretion will in turn cause further flow reductions in the area, yielding further accretion.



EXPECTED SHOALING OF THE CHANNELS AND TIDAL FLATS AFFECTED BY THE CROSS-DAM

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG.D.8

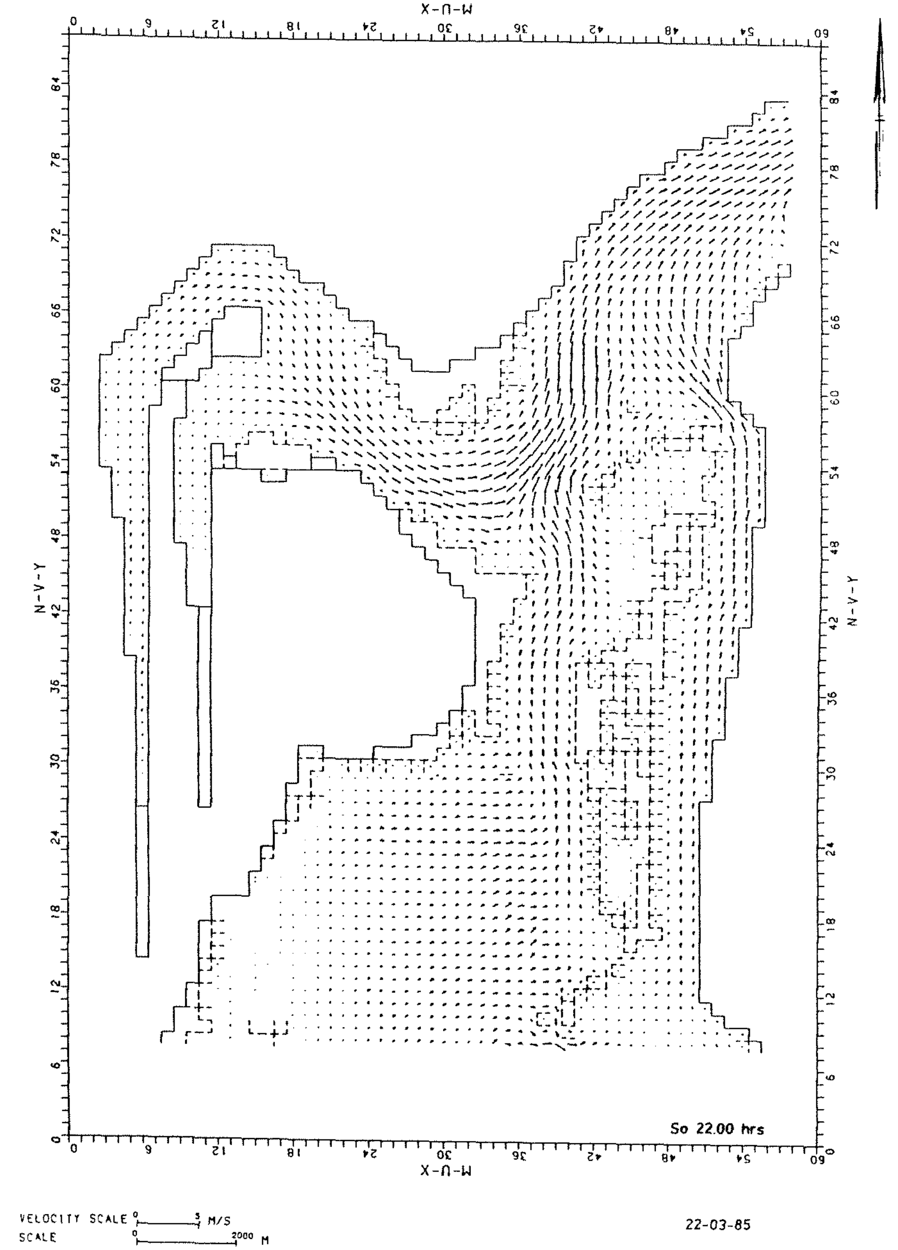


FLOW PATTERNS NEAR WESTERN CHANNEL
BEFORE CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.9a

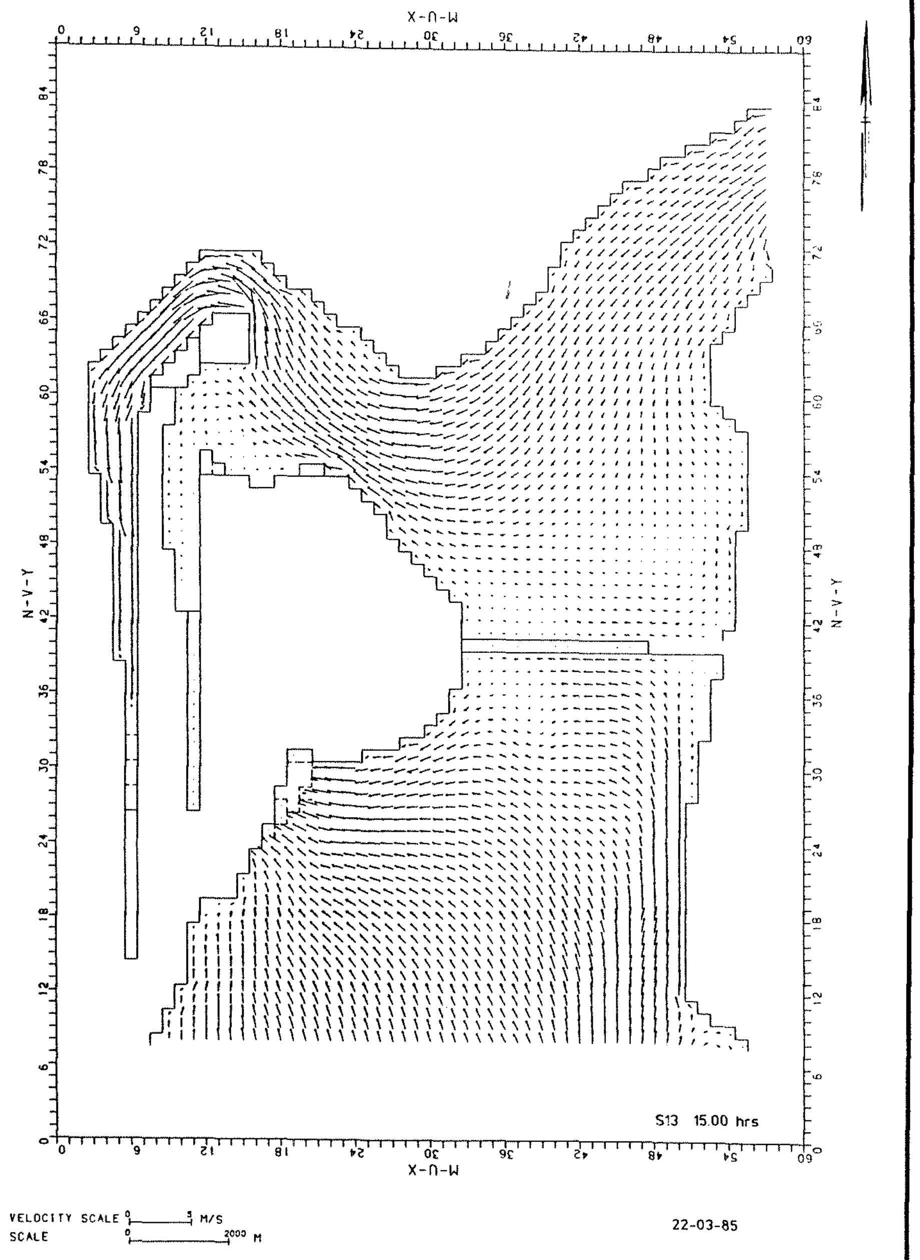


FLOW PATTERNS NEAR WESTERN CHANNEL
BEFORE CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG D.9b

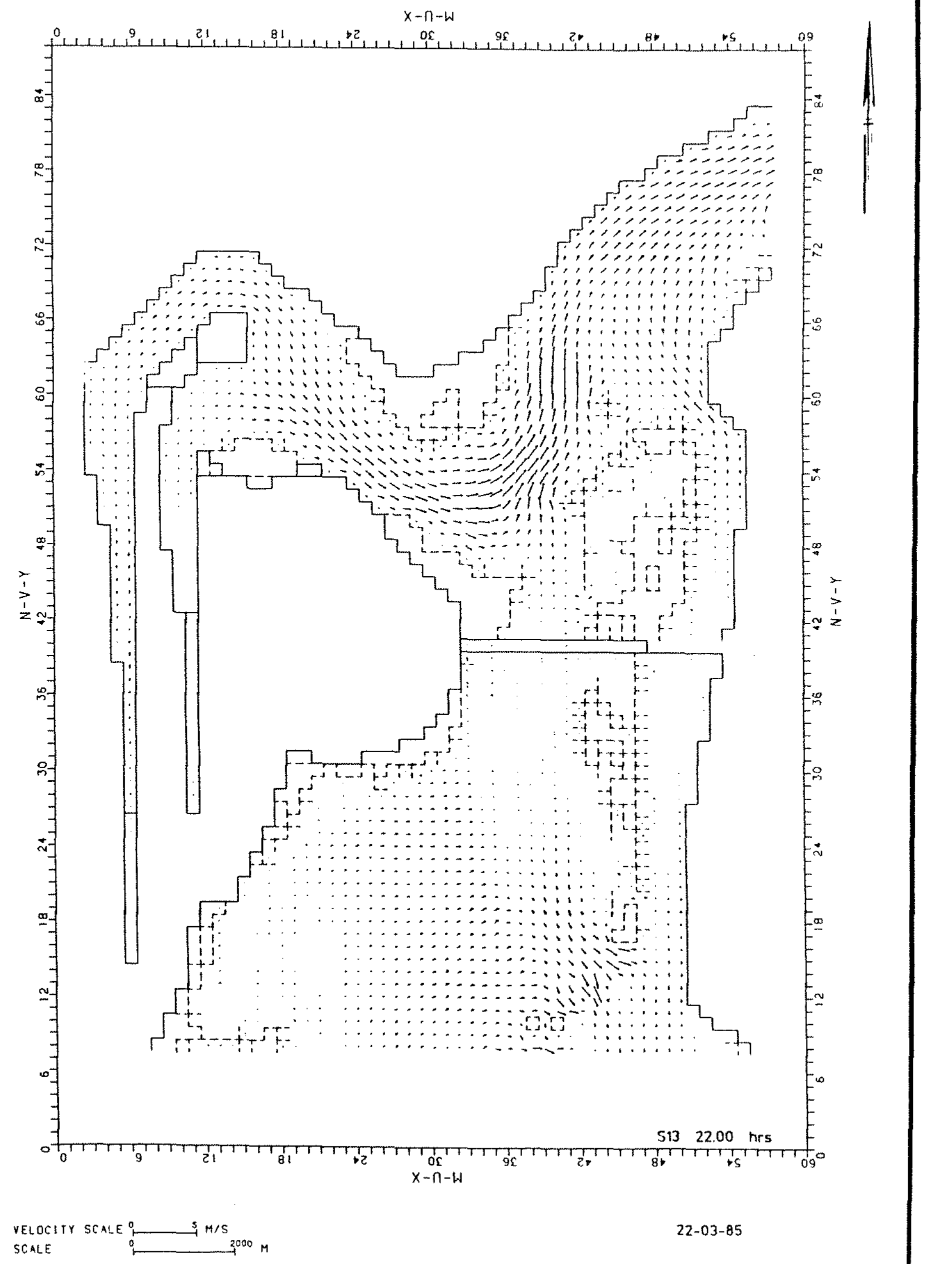


FLOW PATTERNS NEAR WESTERN CHANNEL AFTER CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG D.9c

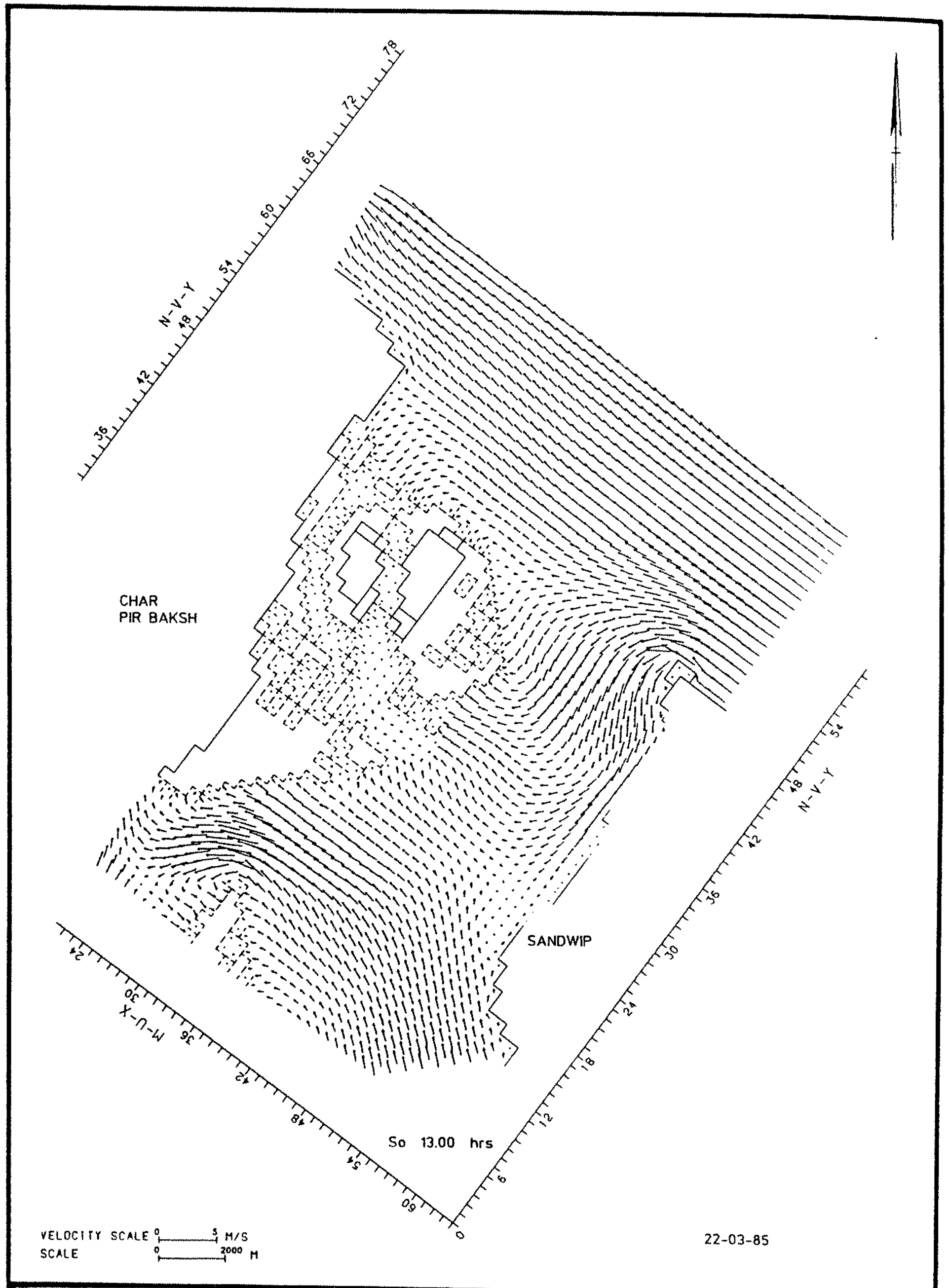


FLOW PATTERNS NEAR WESTERN CHANNEL AFTER CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG D9d

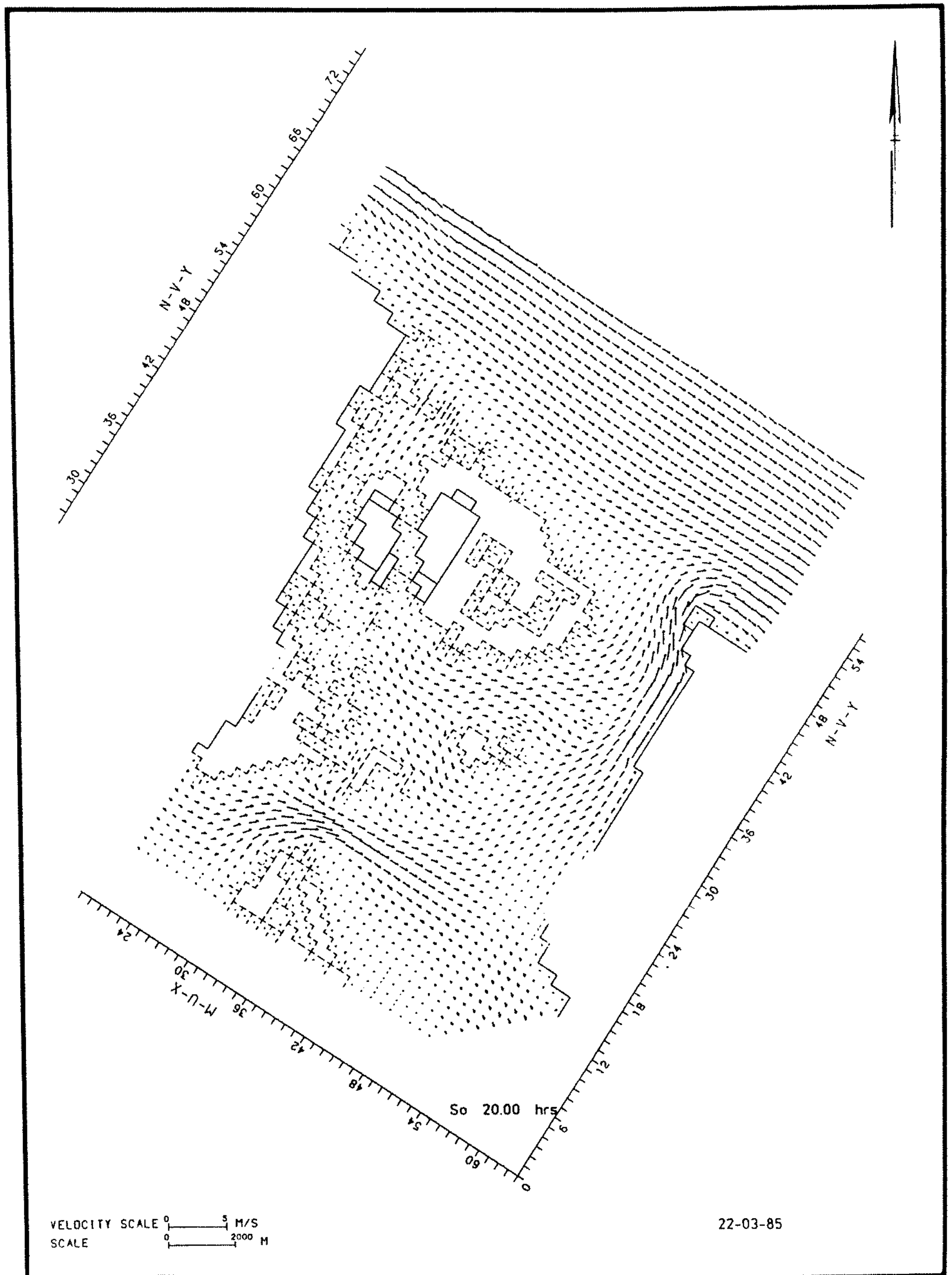


FLOW PATTERNS NEAR EASTERN CHANNEL BEFORE CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.10a

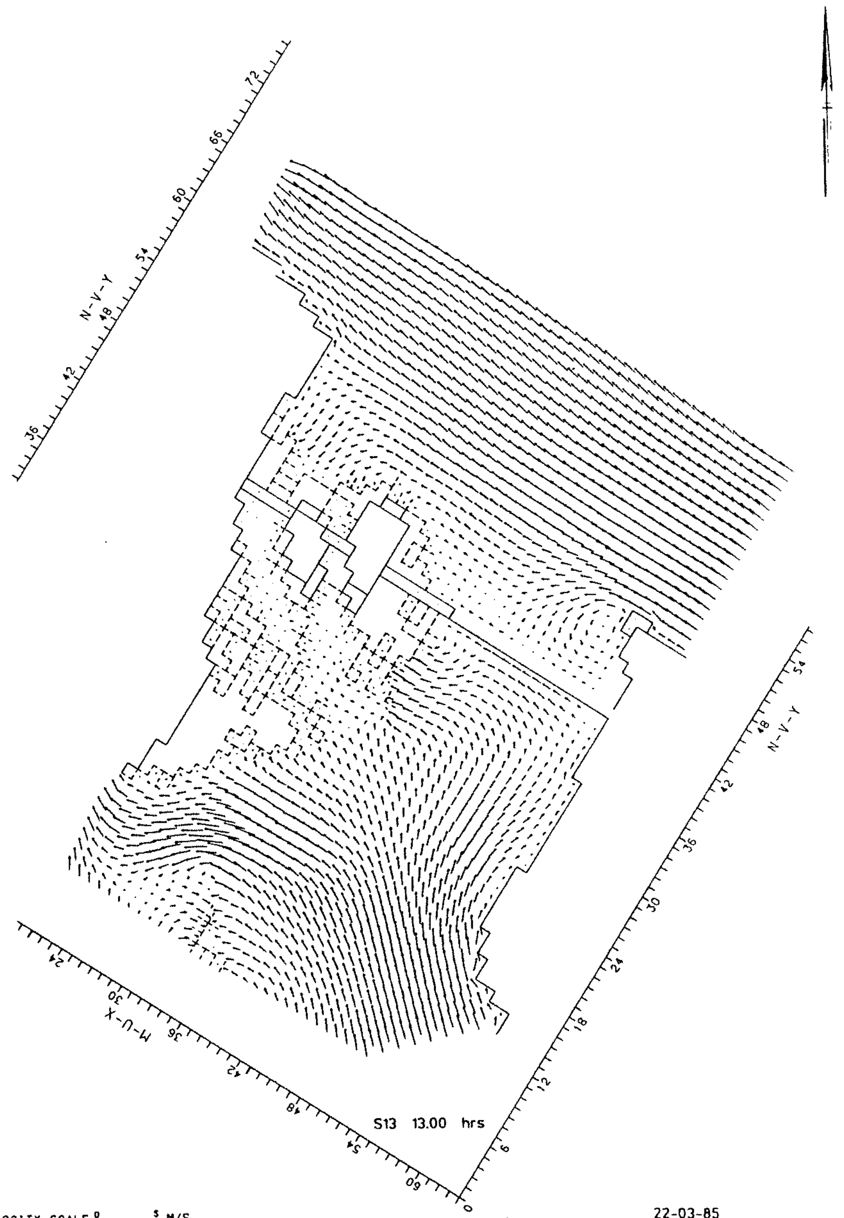


FLOW PATTERNS NEAR EASTERN CHANNEL BEFORE CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.10b



VELOCITY SCALE 0 — 5 M/S
SCALE 0 — 2000 M

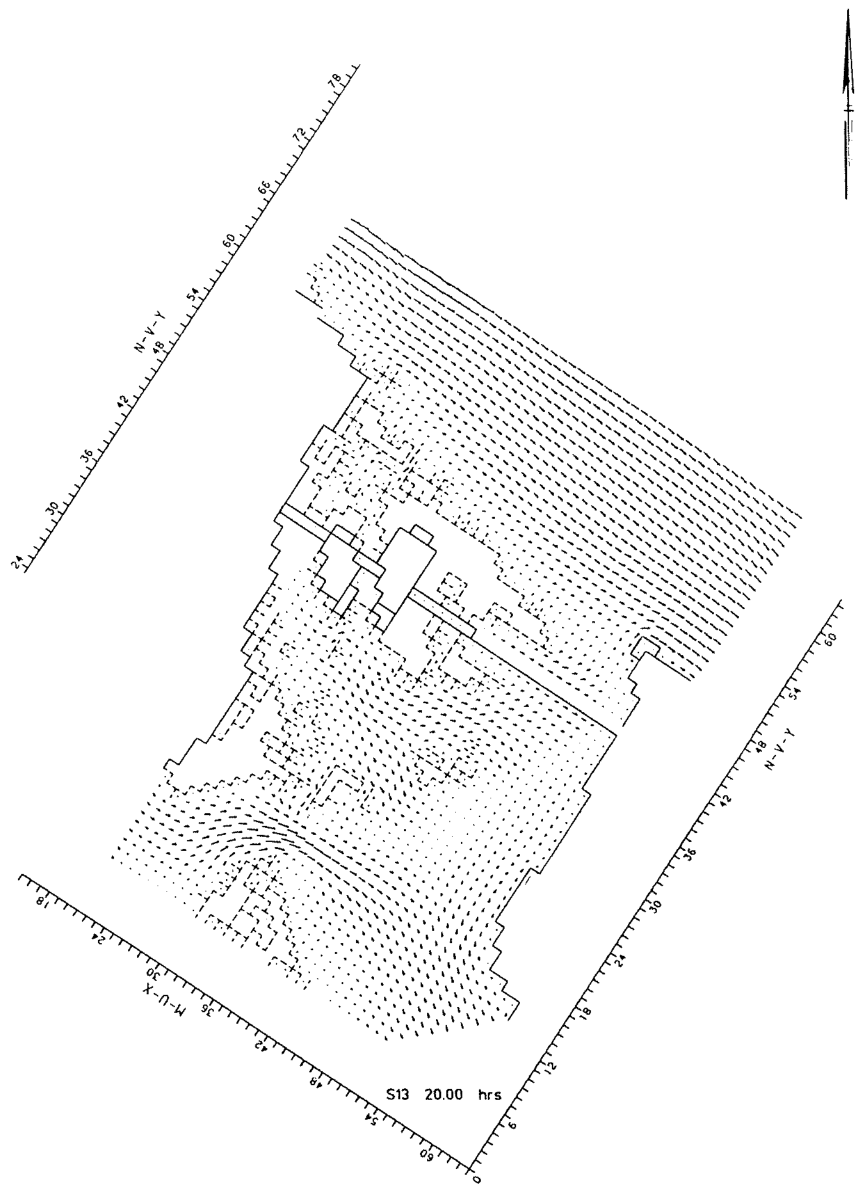
22-03-85

FLOW PATTERNS NEAR EASTERN CHANNEL
AFTER CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.10c



VELOCITY SCALE 0 — 5 M/S
SCALE 0 — 2000 M

22-03-85

FLOW PATTERNS NEAR EASTERN CHANNEL
AFTER CLOSURE

SPRINGTIDE

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG D.10d

In practice, the initial rate of accretion will not be maintained. Similar to the shoaling approach, the following equation for the accretion history can be derived:

$$A = A_o\ [1 - \exp.(- t/\tau)] \qquad (D.5)$$

Or after rewriting:

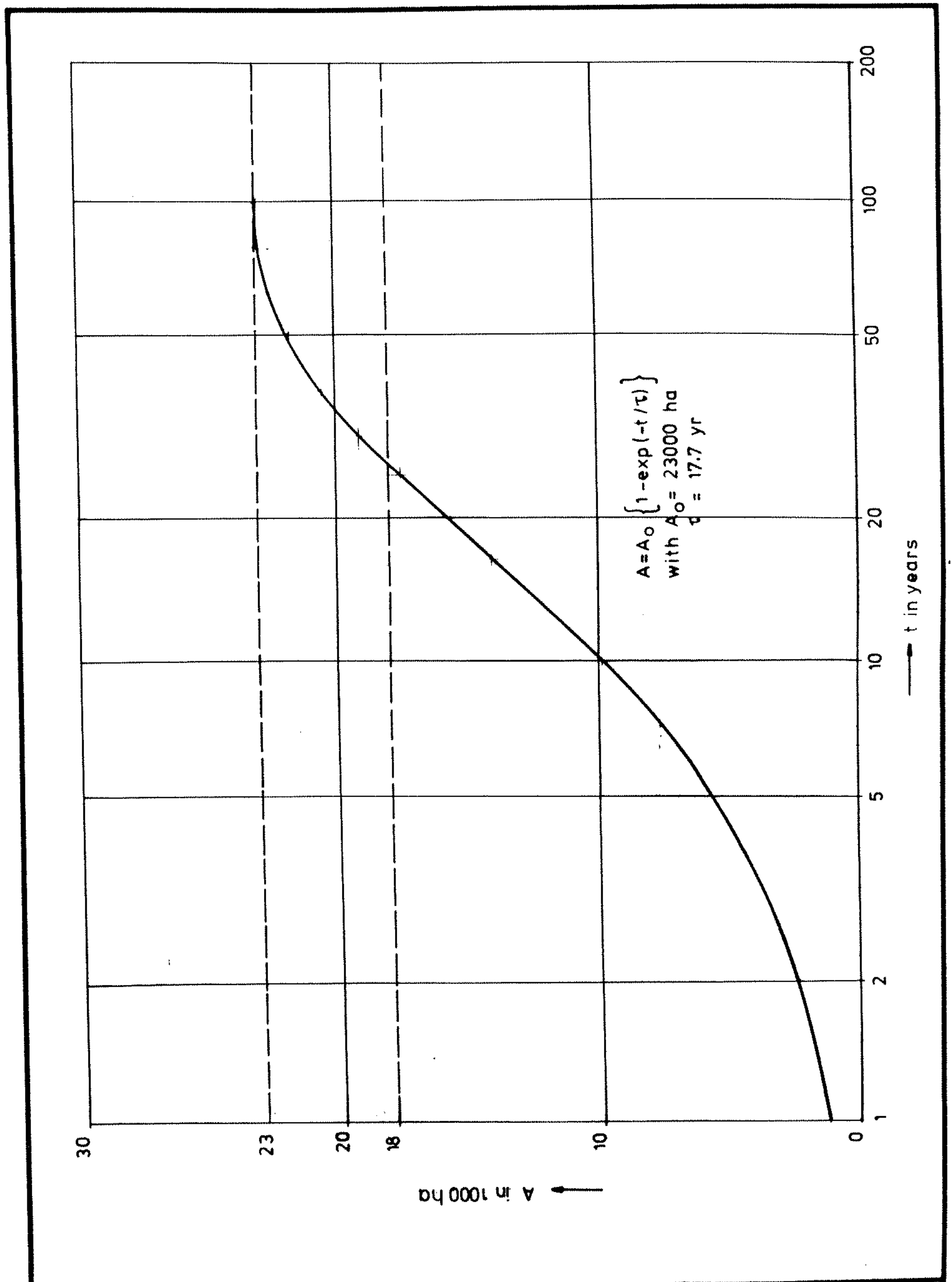
$$t = \tau\ \ln[A_o/(A_o - A)] \qquad (D.6)$$

where A : area of accreted land (e.g. in ha) with a level exceeding PD + 4.5 m at time t (years)
A_o : total area of ultimate accretion (in the same unit as A)
τ : accretion time in years if total accretion occurred at initial rate.

A_o was determined at about 23 000 ha based on the geometry of the area and the predicted hydraulic conditions. This area, and the initial rate of accretion yield τ equal to about 17.7 years. It is self-evident that these values of A_o and I are approximate. The same applies to Figure D.11 which shows the expected accretion with time, based on the figures.

Finally, tentative accretion patterns have been roughly determined as a basis for the economic evaluation of the land reclamation, and as a boundary condition for the cross-dam. Some results are shown in Figure D.12. The dimensions of the main channels have been based on Equation D.1 with a coefficient of 65.

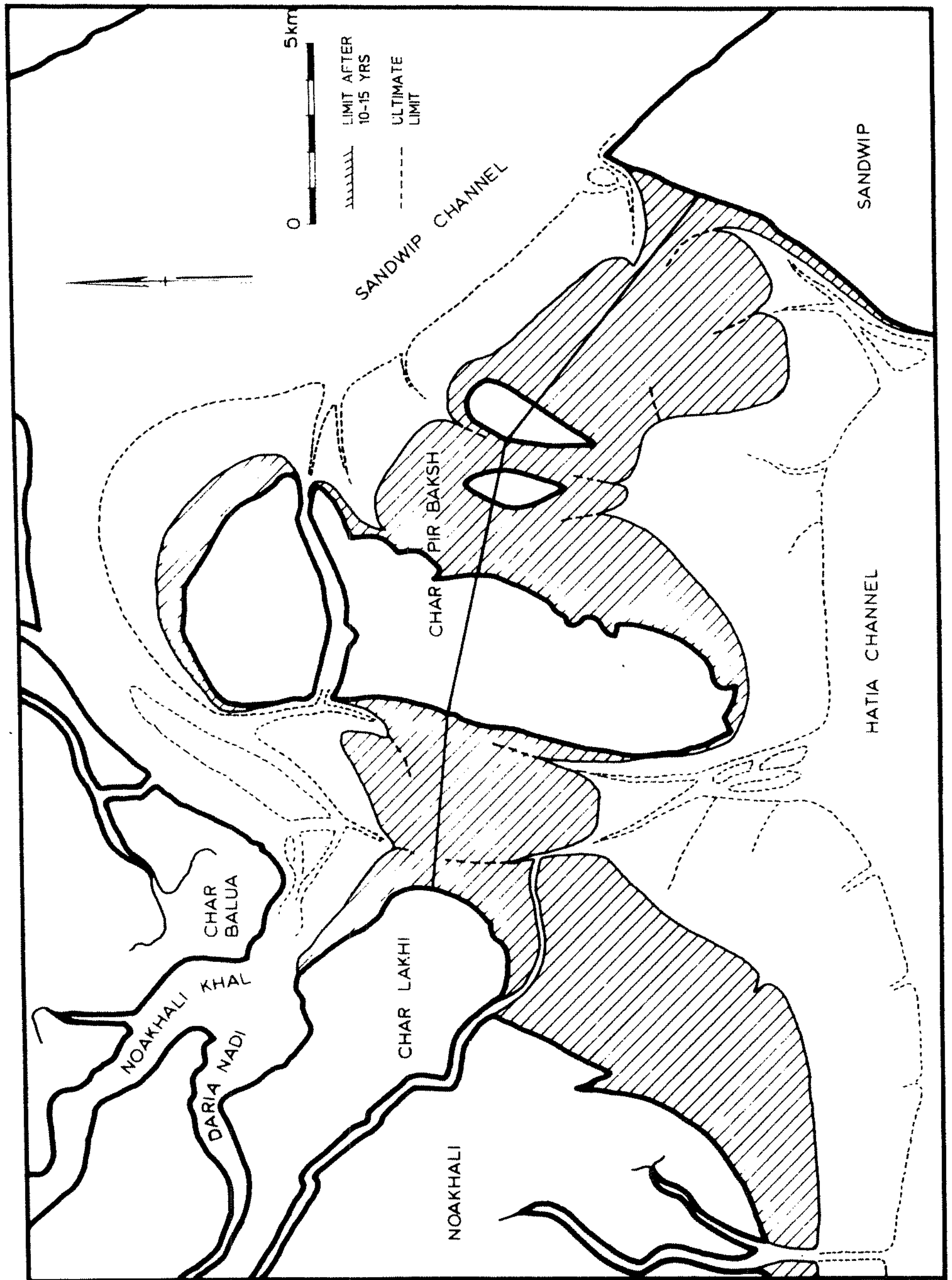
These developments only occur when the dam is of sufficient height to prevent any flow under normal conditions. Incidental overflow over the dam for a short period under extreme conditions e.g. during a cyclone with high water-spring-tide, will not significantly hamper the accretion process.



EXPECTED TIME-HISTORY OF ACCRETION
DUE TO SANDWIP CROSS-DAM

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.11



TENTATIVE PICTURE OF ACCRETION OF NEW LAND 10-15 YEARS AFTER COMPLETION OF SANDWIP CROSS-DAM

SANDWIP CROSS-DAM FEASIBILITY STUDY

FIG. D.12

D.6 Effects on erosion and sedimentation elsewhere

D.6.1 General

Generally speaking, the cross-dam and subsequent accretion on either side of the dam have a rather local effect on the water-levels and currents and, therefore, on the morphological processes. Although the current velocities in the channels between Char Lakhi and Sandwip island can be very high, the volumes of water passing through these channels are negligible in comparison to the tidal volumes passing through the Sandwip and Hatia channels. Therefore, the cross-dam will have very little effect on the flow in these channels, and will have only the local effect of stopping the erosion of Sandwip island and Char Pir Baksh, and of accretion on either side of the dam between Sandwip island and the Noakhali mainland.

When the cross-dams 1 and 2 were constructed across the Lower Meghna river branch (the Bamni river), this river was already shifting to the present course into the Shabazpur channel. This natural process caused erosion of Bhola and Hatia islands because the upland discharges enlarged the cross-sections of the new channels. The cross-dams 1 and 2 were seen as the cause of this erosion which, however, is not the case, since the change of the upland discharge channels was a natural process, which would also have taken place without these cross-dams.

The Sandwip cross-dam should not be compared with the cross-dams 1 and 2, because it does not block any upland discharges and is not related to any changes in the major upland discharge channels.

As stated in the introduction to this Annex (chapter D.1), the effects of the construction of the cross-dam, and the subsequent accretion on either side of the dam on the water-levels and currents elsewhere in the area have been investigated by means of two-dimensional mathematical models based on the computer program WAQUA, and a one-dimensional model based on the computer program NETFLOW. These models have been developed using the most recent bathymetric surveys and maps, and the changes in the geometry due to the cross-dam itself, and the expected subsequent accretion have been superimposed on the otherwise fixed bed. The model computations confirm what has been said above, that the dam and accretion of new land will have only a local effect on the water-levels and currents.

At some distance from the dam, the changes in tidal volumes appear to be limited to only a few percent, with the exception of the navigation route south and west of Sandwip island, where reduction of the tidal volumes of up to 9 percent have been computed. To translate these changes into morphological adaptations the following interpretation may be used: changes in tidal volumes smaller than 2.5 percent will not cause any significant morphological changes; between 2.5 and 5 percent some minor changes may occur; changes above 5 percent may lead to noticeable adaptations, either sedimentation or erosion. The discussion of some of the locations which are considered to be sensitive, presented in the next paragraphs, is based on this interpretation.

The major impact of the cross-dam and subsequent accretion on the water-levels is summarized in Table D.3.

Table D.3 - Changes in water-levels under spring-tide conditions due to the construction of Sandwip cross-dam (m)

Channel	Location	Without acretion		With accretion	
		High water	Low water	High water	Low water
Sandwip	Char Pir Baksh-West(dam)	+ 0.5	- 0.8	+ 0.25	- 0.5
	Feni Dam	+ (0.1-0.15)	- (0-0.15)	+ (0.3-0.5)	- 0.05-0.2)
	Sandwip-North	+ 0.1	- 0.15	+ 0.3	- 0.2
Hatia	Char Pir Baksh-West(dam)	- 0.4	+ 0.4	-	-
	Sandwip-West	- 0.05	+ 0.1	0.1	+ 0
	Noakhali	- 0.05	+ 0.1	+ 0	+ 0

It goes without saying that the quantitative results of the computations made with the mathematical models should not be seen as absolute figures. Nevertheless, the results are reliable enough to draw conclusions with regard to tendencies and order of magnitude, and allow a prediction with reasonable confidence, of the changes, if any, to be expected in the present morphological processes in the area, due to the construction of the cross-dam. Notwithstanding this, the authorities concerned may consider it advisable to monitor the areas for which they are responsible.

D.6.2 Bhola and Hatia

The two-dimensional model computations made in 1983/84 showed that no significant effects could be observed in the Shabazpur channel, while the current velocities along the north coast of Hatia appear to remain virtually the same in all situations. This is confirmed by the one-dimensional computations carried out in 1987 which indicate only minor changes (mostly reductions) in tidal volumes at these locations.

This means that no adverse effects are to be expected due to the cross-dam. On the other hand, unless protective measures are taken, the present erosion of the east coast of Bhola, and of the north coast of Hatia probably due to the gradual enlarging of the bend of the tidal channel (LRP, 1984a, p. 9), must be expected to continue.

D.6.3 Hatia channel

The navigation channel from Chittagong to Chandpur and further upriver is regularly monitored and shifted to the most navigable natural channels. Unless the natural channels are fixed by bank protection and river training works, this process will continue in the future. At present, the main navigation channel on this route is located in Hatia channel and goes from Hatia North to Bhola and then north.

The computations of the mathematical model show that after the construction of the cross-dam and the subsequent accretion near the dam,

the tidal volumes in the navigation route between Sandwip South and Hatia North will ultimately be reduced by 6 to 8 percent. This could result in a depth reduction of about 0.3m. Generally this will cause no problems as sufficient depth is available, but in the area west of Sandwip this may cause problems. On the other hand, due to the absence of strong currents in the cross-channels on both sides of Char Pir Baksh after dam construction, the situation may even improve. The channel there will have an uninterrupted bank, and is likely to develop a continuous profile without hazardous shoals, and have sufficient depths for navigation with draughts up to 3 m.

Erosion of the western coast of Sandwip will continue but may be favourably effected by the slightly reduced current velocities. In so far as this erosion is caused by wave attack, this will continue and will not be effected one way or the other by the construction of the cross-dam.

D.6.4 Sandwip channel, Chittagong port

After the construction of the cross-dam, the tidal volumes will slightly increase due to the extra reflection of the tidal wave. In the northern part of the channel the accretion near the dam will reduce this effect and ultimately even a slight decrease, compared with the present situation, may occur. The computations of the mathematical model made in 1987 show, however, that in the reach between Sandwip island and the Karnaphuli river, the tidal volumes will increase even further due to the land accretion.

Due to the sedimentation near the cross-dam, a residual sediment transport in a northerly direction will be created in the Sandwip channel, which will carry any material scoured from this channel rather north than south. Eventually the Sandwip channel may silt-up further in a southerly direction, but this will be a very slow process, not affected by the construction of the Sandwip cross-dam in any significant way.

The above-mentioned computations for the mathematical model and considerations lead to the following conclusions with regard to locations in the Sandwip channel which are considered to be sensitive:

Chittagong port: Because the velocities after the accretion near the cross-dam show a tendency to become higher in the area, and because of the expected northern residual sediment transport, no siltation at all is expected at Outer Anchorage.

The tidal propagation, flow and sediment regime of the Karnaphuli river only depends on the up-river discharge, governed by Kaptai dam, and the tide levels at the mouth of the river. The latter will not change due to the construction of the cross-dam. Hence, no impact on the regime of the Karnaphuli river is to be expected.

Chittagong mainland and Char Balua. For the Chittagong mainland, the same conditions apply as for Sandwip East; no noticeable changes which can be ascribed to the construction of the cross-dam can expected. As Section D.4.1 shows, the mean tidal volume passing the channel between Char Pir Baksh and Char Balua is expected to be initially reduced by about 10 million m^3, and by more after accretion of new land at the dam. The dam will therefore have no negative effect on the foreshore development of Char Balua.

D.6.5 Noakhali khal and Daria nadi

Concern has been expressed that the construction of the cross-dam and the subsequent accretion near the dam will impede the drainage function of the Noakhali khal and Daria nadi. It has also been suggested that an alternative alignment for the western closure dam, running from the northern coast of Char Pir Baksh to the mainland, in an approximately northern direction, would have a less harmful effect on this drainage function than the alignment proposed in this report. Both alignments have been investigated in the study, and from a hydraulic point of view there is a very strong preference for the proposed alignment, as has been explained in chapter C.3. The morphological and drainage aspects will be discussed in this paragraph.

The drainage function of the Noakhali khal is often overestimated. Flow measurements by the LRP indiate that at Sonapur the flow of the Noakhali khal is not directed to the south, but instead northward to Chowmauhani from where it drains through the WAPDA khal via the Rhamat khal regulator into the Lower Meghna. The main reason for this is the silting up of the mouth of the Noakhali khal, resulting in a year by year reduction of its size and drainage capacity. The LRP is investigating the effect of this siltation on the future drainage of the Noakhali khal catchment area.

If the cross-dam is built on the alignment proposed in this report, the Noakhali khal will discharge into the Sandwip channel. As stated in section D.6.4, initially the tidal volumes in this channel will slightly increase due to the extra reflection of the tidal wave, which will probably cause some erosion of Noakhali khal and thus improve its drainage capacity. In the long run however, the channels connecting the Noakhali khal with the Sandwip channel will reduce in size with the increasing land accretion at the dam, west of the Char Pir Baksh. This process will stop when the accretion stops, leaving a channel with a profile adapted to the then prevailing tidal volumes. The situation with regard to the drainage function of the Noakhali khal will then be comparable to the present situation, described above, and the subject of a study by the LRP.

If the dam is constructed along the suggested northern alignment, the Noakhali khal will have to discharge into the Hatia channel. Eventually the area between the mainland and Char Pir Baksh, up to this closure dam, will be silted up, also leaving a channel with a profile adapted to the then prevailing tidal volumes. There is no reason to expect that this situation will be more favourable than that described above, with regard to the drainage function of the Noakhali khal.

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